City of Loganville
Storm Water Design Manual

Stormwater Design Manual
City of Loganville

Table of Contents

1 Introduction

1.1 Purpose of Manual

The purpose of this manual is to address the impacts of urban development and stormwater runoff on the environment. Chapter 2, of the Georgia Stormwater Management Design Manual states, “The growth of Georgia’s towns, cities, and suburbs has profoundly altered the natural drainage system and flow of water in the environment.” The U.S. Congress passed the Clean Water Act in 1972 with a stated objective to restore and maintain the chemical, physical, and biological integrity of the nation’s waters through point source and non-point source controls. The method to achieve this restoration process is through the implementation of “Stormwater Management Practices” (SMPs). The City of Loganville has decided to take a proactive approach to watershed management by addressing both Stormwater quantity and quality issues.

Increased emphasis on water quality, resource protection needs, and increased SMP maintenance costs have all contributed to the improvements regarding stormwater management within the City of Loganville. The City of Loganville Stormwater Design Manual is an effort to achieve needed improvements for managing stormwater runoff.

Additionally, the changes shall produce smaller less obtrusive facilities that are more aesthetic and less burdensome on those responsible for long-term maintenance and performance. In summary, the purpose of this manual is to:

- Protect the State waters from adverse impacts from stormwater runoff,
- To provide design guidance on the most effective structural and non-structural SMPs for development sites, and
- To improve the quality of SMPs that are constructed in the City of Loganville, specifically with regard to performance, longevity, safety, ease of maintenance, community acceptance, and environmental benefit.

1.2 Impact of Stormwater Runoff

Urban development has a profound influence on the quality of the City’s waters. Development affects the local hydrologic cycle (see Figure 1-1). The hydrology of a site changes during the initial clearing and grading that occur during construction. Trees, meadow grasses, and agricultural crops that had intercepted and absorbed rainfall are removed and natural depressions that had temporarily ponded water are graded to a uniform slope. Cleared and graded sites erode, are often severely compacted, and can no longer prevent rainfall from being rapidly converted into stormwater runoff.

The situation worsens after construction. Rooftops, roads, parking lots, driveways and other impervious surfaces no longer allow rainfall to soak into the ground. Consequently, most rainfall is converted directly to stormwater runoff. The runoff coefficient expresses the fraction of rainfall volume that is converted into stormwater runoff. As can be seen, the volume of stormwater runoff increases sharply with impervious cover. For example, a one-acre parking lot can produce 16 times more stormwater runoff than a one-acre meadow each year (Schueler, 1994).

The increase in stormwater runoff can be too much for the existing natural drainage system to handle. As a result, the natural drainage system is often “improved” to rapidly collect runoff and quickly convey it away (using curb and gutter, enclosed storm sewers, and lined channels). The stormwater runoff is subsequently discharged to downstream waters such as streams, reservoirs, lakes or estuaries.

1.2.1 Declining Water Quality

Impervious surfaces accumulate pollutants deposited from the atmosphere, leaked from vehicles, or windblown from adjacent areas. During storm events, these pollutants quickly wash off and are rapidly delivered to downstream waters. Some common pollutants found in urban stormwater runoff are profiled in Table 1.1 and include:

*Nutrient*s. Urban runoff has elevated concentrations of both phosphorus and nitrogen, which can enrich streams, lakes, reservoirs and estuaries (known as eutrophication). Excess nutrients promote algal growth that blocks sunlight from reaching underwater grasses and depletes oxygen in bottom waters. Urban runoff has been identified as a key and controllable source.

Suspended solids. Sources of sediment include wash off of particles that are deposited on impervious surfaces and the erosion of stream banks and construction sites. Both suspended and deposited sediments can have adverse effects on aquatic life in streams, lakes and estuaries. Sediments also transport other attached pollutants.

*Organic Carbo*n. Organic matter, washed from impervious surfaces during storms, can present a problem in slower moving downstream waters. As organic matter decomposes, it can deplete dissolved oxygen in lakes and tidal waters. Low levels of oxygen in the water can have an adverse impact on aquatic life.

*Bacteri*a. Bacteria levels in stormwater runoff routinely exceed public health standards for water contact recreation. Stormwater runoff can also lead to the closure of adjacent shellfish beds and swimming beaches and may increase the cost of treating drinking water at water supply reservoirs.

*Hydrocarbon*s. Vehicles leak oil and grease that contain a wide array of hydrocarbon compounds, some of which can be toxic at low concentrations to aquatic life.

*Trace Metal*s. Cadmium, copper, lead and zinc are routinely found in stormwater runoff. These metals can be toxic to aquatic life at certain concentrations and can also accumulate in the sediments of streams and lakes.

Pesticides. A modest number of currently used and recently banned insecticides and herbicides have been detected in urban stream flow at concentrations that approach or exceed toxicity thresholds for aquatic life.

*Chloride*s. Salts that are applied to roads and parking lots in the winter months appear in stormwater runoff and melt-water at much higher concentrations than many freshwater organisms can tolerate.

*Thermal Impact*s. Impervious surfaces may increase temperature in receiving waters, adversely impacting aquatic life that requires cold and cool water conditions (e.g., trout).

*Trash and Debri*s. Considerable quantities of trash and debris are washed through storm drain networks. The trash and debris accumulate in streams and lakes and detract from their natural beauty.

1.2.2 Degradation of Stream Channels

Stormwater runoff is a powerful force that influences the geometry of streams. After development, both the frequency and magnitude of storm flows increase dramatically. Consequently, urban stream channels experience more bankfull and sub-bankfull flow events each year than they had prior to development.

As a result, the streambed and banks are exposed to highly erosive flows more frequently and for longer periods. Streams typically respond to this change by increasing cross-sectional area to handle the more frequent and erosive flows either by channel widening or down cutting, or both. This results in a highly unstable phase where the stream experiences severe bank erosion and habitat degradation. In this phase, the stream often experiences some of the following changes:

- Rapid stream widening
- Increased stream bank and channel erosion
- Decline in stream substrate quality
- Loss of pool/riffle structure in the stream channel
- Degradation of stream habitat structure

The decline in the physical habitat of the stream, coupled with lower base flows and higher stormwater pollutant loads, has a severe impact on the aquatic community. Recent research has shown the following changes in stream ecology:

- Decline in aquatic insect and freshwater mussel diversity
- Decline in fish diversity
- Degradation of aquatic habitat

Traditionally, the City of Loganville has attempted to provide some measure of channel protection by imposing the two-year storm peak discharge control requirement, which requires that the discharge from the two-year post development peak rates be reduced to pre development levels. However, recent research and experience indicate that the two-year peak discharge criterion is not capable of protecting downstream channels from erosion. In some cases, controlling the two-year storm may actually accelerate stream bank erosion because it exposes the channel to a longer duration of erosive flows than it would have otherwise received.

1.2.3 Increased Overbank Flooding

Flow events that exceed the capacity of the stream channel spill out into adjacent floodplains. These are termed “overbank” floods and can damage property and downstream drainage structures. While some overbank flooding is inevitable and even desirable, the historical goal of drainage design in most of the City of Loganville has been to maintain pre development peak discharge rates for both the two and ten-year frequency storms after development, thus keeping the level of overbank flooding the same over time. This prevents costly damage or maintenance for culverts, drainage structures, and swales.

Overbank floods are ranked in terms of their statistical return frequency. For example, a flood that has a 50% chance of occurring in any given year is termed a “two-year” flood. The two-year storm is also known as the “bankfull flood,” as researchers have demonstrated that most natural stream channels in the City have just enough capacity to handle the two-year flood before spilling into the floodplain. In Loganville, about 3.8 inches of rain in a 24-hour period produces a two-year or bankfull flood. This rainfall depth is termed the two-year design storm. Similarly, a flood that has a 10% chance of occurring in any given year is termed a “ten-year flood." A ten-year flood occurs when a storm event produces about 5.8 inches of rain in a 24-hour period. Under traditional engineering practice, most channels and storm drains in Loganville are designed with enough capacity to safely pass the peak discharge from the ten-year design storm.

Urban development increases the peak discharge rate associated with a given design storm because impervious surfaces generate greater runoff volumes and drainage systems deliver it more rapidly to a stream.

1.2.4 Expansion of the Floodplain

The level areas bordering streams and rivers are known as floodplains. Operationally, the floodplain is usually defined as the land area within the limits of the 100-year storm flow water elevation. The 100-year storm has a 1% chance of occurring in any given year and typically serves as the basis for controlling development in the State and establishing insurance rates by the Federal Emergency Management Agency (FEMA). In Loganville, a 100-year flood occurs after about 8 inches of rainfall in a 24-hour period (e.g., the 100-year storm). These floods can be very destructive and can pose a threat to property and human life. Floodplains are natural flood storage areas and help to attenuate downstream flooding.

Floodplains are very important habitat areas, encompassing riparian forests, wetlands, and wildlife corridors. Consequently, all local jurisdictions in Loganville restrict or even prohibit new development within the 100-year floodplain to prevent flood hazards and conserve habitats. Nevertheless, prior development that has occurred in the floodplain remains subject to periodic flooding during these storms. As with overbank floods, development sharply increases the peak discharge rate associated with the 100-year design storm. As a consequence, the elevation of a stream’s 100-year floodplain becomes higher and the boundaries of its floodplain expand (see Figure 1-2). In some instances, property and structures that had not previously been subject to flooding are now at risk. Additionally, such a shift in a floodplain’s hydrology can degrade wetlands and forest habitats.

1.3 General Performance Standards

To prevent adverse impacts of stormwater runoff, the City of Loganville has developed nine performance standards that must be met at development sites. These standards apply to any construction activity disturbing 5,000 or more square feet of earth. The following development activities are exempt from these performance standards in the City of Loganville:

1. Additions or modifications to existing single-family structures;
2. Developments that do not disturb more than 5000 square feet of land; or
3. Agricultural land management activities.
4. An individual single house. (Single-family houses that are part of a subdivision or phased development project shall not be exempt from the recommended requirements.)

Development in critical or sensitive areas may be subject to additional performance requirements, or may need to utilize or restrict certain structural controls in order to protect a special resource or address certain water quality or structural problems identified for a drainage area. The following performance standards shall be addressed at all sites where stormwater management is required:

Performance Goal #1

Site designs shall strive to preserve and utilize natural drainage systems and reduce the generation of additional stormwater runoff to the maximum extent practicable.

▸ Encourage use of better site design practices and techniques to reduce impervious areas, hydrologically disconnect impervious areas so that they drain to vegetated areas, utilize natural site features for stormwater management, and incorporate onsite bioretention areas through landscaping practices.

▸ Prevent unnecessary stripping of vegetation and loss of soils, especially adjacent to lakes, streams, watercourses, and wetlands.

▸ Where feasible, conserve forested and undisturbed vegetated areas, and establish and maintain riparian buffers.

Performance Goal #2

Stormwater runoff generated from new development shall be adequately treated and controlled prior to discharge into a jurisdictional wetland or waters of the state.

For all new development sites, stormwater management systems (which can include both structural stormwater controls and better site design practices) shall be designed to remove 80% of the average post-development total suspended solids (TSS). Acceptable structural controls shall be limited to those practices that have a demonstrated ability to meet this performance criterion. It is presumed that a structural control complies with this performance standard if it is:

▸ Sized to capture the prescribed water quality treatment volume, which is defined as the volume resulting from the first 1.2 inches of runoff from a site;

- Designed to meet the design requirements presented in this plan;
- Constructed properly; and
- Inspected and maintained on a regular basis.

Performance Goal #3

Local communities shall require, as necessary, additional or site-specific management measures to control and treat stormwater runoff from certain types of development and areas draining to sensitive receiving waters.

- Redevelopment, defined as any construction, alteration or improvement exceeding 5,000 square feet in areas where existing land use is high density commercial, industrial, institutional, or multi-family residential, shall be governed by special stormwater sizing criteria depending on the amount of increase or decrease in impervious area created by the redevelopment.
- Where onsite stormwater management facilities are not practical, fee in-lieu of treatment may be required.
- Stormwater discharges from land uses or activities with higher potential pollutant loadings, defined as hotspots (ex. gas stations, convenience stores, auto-recycling areas, etc.), may require the use of specific structural controls and pollution prevention practices. In addition, stormwater from a hotspot land use shall not be infiltrated without effective pretreatment. For example, hotspots might be required to prepare and implement stormwater pollution prevention plans that minimize pollutant generation and contact of rainfall with pollutants.
- Stormwater discharges to critical areas with sensitive resources (i.e., cold water fisheries, shellfish beds, swimming beaches, recharge areas, water supply sources, river corridors) may be subject to additional performance criteria, or may need to utilize or restrict certain structural controls. For example, if phosphorus loading is a receiving water concern, specific phosphorus load reduction mandates may be warranted.

Adequately treated means that the designated water quality volume has been treated through one or more of the approved stormwater controls and/or site practices that are detailed in this plan. The 80% removal goal is a management measure developed by the Environmental Protection Agency (EPA). It was selected by the EPA for the following factors:

- Removal of 80% is assumed to control heavy metals, phosphorus, and other pollutants
- Data show that certain structural controls, when properly designed and maintained, can meet the 80% removal performance level.

Performance Goal #4

Stream channel protection shall be provided by adopting three general approaches:

- Upland sources control and detention
- Bank protection measures such as energy dissipation and velocity control; and
- Riparian corridor preservation and conservation.

This goal may not be necessary for sites draining to large water bodies such as lakes, or major rivers.

- Provide extended detention storage for the 1-year frequency storm event.
- Establish well-forested and undisturbed vegetated riparian buffers. The buffers shall be a minimum of 25 feet.
- It is recommended that 100-foot vegetated buffers be established, where feasible.
- Energy dissipation shall be provided at all stormwater outfalls to ensure discharges exit at non-erosive velocities.

Performance Goal #5

Overbank flood protection shall be provided for by all sites discharging water to a stream or river.

- Provide control of the post-development peak discharge rate to predevelopment rate for 25-year return frequency storm event.

Performance Goal #6

All habitable structures and major transportation arteries (roads, railroads, etc.) shall be protected from flooding to at least the 100-year flood level for the expected life of the structure from all flooding sources, major and minor.

- The increase in runoff volumes and peaks shall be minimized and kept to predevelopment levels as possible.

- The full *build-out* floodplain shall be established and development restricted in these areas. Already existing flood susceptible development shall be acquired or protected from flood damage.

- Citizens shall be informed and warned about the flooding potential of property prior to purchase and development.

Performance Goal #7

Local communities shall require effective short and long-term maintenance of all of the drainage system and structural stormwater controls.

- All structural controls and stormwater facilities shall have an enforceable operation and maintenance agreement to ensure the system functions as designed.

- The condition of the drainage system shall be known and maintenance decisions shall be proactively made on the basis of inspections rather than solely on the basis of complaints of flooding, erosion, or pollution.

Performance Goal #8

Regional stormwater management facilities shall be evaluated as an alternative for onsite water quantity and water quality controls. Regional stormwater management refers to facilities designed to manage runoff from multiple projects and/or properties through a local jurisdiction-sponsored program, where the individual properties assist in the financing of the facility, and the requirement for onsite controls is either eliminated or reduced.

- Master plans and hydraulic and hydrologic models, sufficient to provide regional information, can be used to evaluate regional stormwater facilities.

- Institutional mechanisms supporting the regional concept shall be evaluated.

Performance Goal #9

To the maximum extent practicable, development projects shall strive to implement nonstructural pollutant prevention practices such as material use, exposure, disposal, and recycling controls, spill prevention and cleanup, illegal dumping controls, illicit connection controls, and conservation and preservation measures.

- Require a pollution prevention plan, detailing the use of specific pollution prevention practices, for all development as part of the overall stormwater management concept plan for a site.

- Develop and enforce local ordinances or regulations as necessary to support nonstructural pollution prevention practices and land use controls.

Provide opportunities and institute public education programs to inform citizens and commercials and industrial landowners about relevant pollution prevention practices.

1.4 How to Use the Manual

The City of Loganville Stormwater Design Manual is provided in eight chapters. An overview of each chapter is presented below.

Chapter 1. Introduction.
This chapter presents the purpose of the Stormwater Design Manual. Chapter 1 also presents the provides insight as to the impacts of stormwater runoff, including declining water quality, degradation of stream channels, increased overbank flooding, and floodplain expansion. The stormwater performance goals for the City of Loganville are also presented in this chapter.

Chapter 2. Hydrology
Chapter 2 presents the hydrologic design policies and the preferred design methods and frequencies. This chapter also presents the service charge credits and exemption policy. Example problems are presented to help illustrate the methods described.

Chapter 3 – Storm Drainage Conveyance Systems
Chapter 3 presents guidelines for evaluating roadway features and design criteria as they relate to gutter and inlet hydraulics and storm drain design. Example problems are also presented to illustrate the methods described.

Chapter 4 – Culvert Design
Chapter 4 presents guidelines and criteria for culvert design. Guidelines for velocity limitations, length and slope, headwater limitations, and tailwater considerations are covered in this chapter. This chapter also presents environmental considerations and design nomographs for culvert design. Example problems are presented to help illustrate the methods described.

Chapter 5 – Open Channel Hydraulics
Chapter 5 presents guidelines for open channel hydraulic design. This chapter presents design criteria such as cross slope, side slopes, velocity limitations, Manning's n values, and flow calculations. Example problems are presented to help illustrate the equations and methods provided.

Chapter 6 – Storage Facilities

Chapter 6 presents guidelines for the design of storm drainage storage facilities. Location considerations, release rates, storage, and grading and depth criteria as they pertain to storage facilities are presented within this chapter. Example problems illustrating the methods are presented in this chapter.

Chapter 7 – Energy Dissipation

Chapter 7 presents design guidelines and criteria for designing energy dissipation devices. Erosion hazards and recommended dissipators are presented in this chapter. Example problems for energy dissipation design are also presented in this chapter.

Chapter 8 – Water Quality Stormwater Management Practices

Chapter 8 presents guidelines for water quality stormwater management practices. Structural and non-structural stormwater management practices are presented in this chapter.

Chapter 9 – Limitations

Chapter 9 presents the limitations of the design manual.

1.5 References

Scheuler, T. 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. MWCOG. Washington, DC.

2.1 Hydrologic Design Policies

2.1.1 Hydrologic Method

Many hydrologic methods are available for urban stormwater management. The following methods are recommended and the circumstances for their use are listed in Table 2-1 below. If other methods are used they must first be calibrated to local conditions and tested for accuracy and reliability. In addition, complete source documentation must be submitted for approval.

The following methods have been selected for use in the City of Loganville based on several considerations, including the following:

- Verification of their accuracy in duplicating local hydrologic estimates of a range of design storms.
- Availability of equations, nomographs, and computer programs.
- Use and familiarity with the methods by local municipalities and consulting engineers.

Table 2-1 Recommended Hydrologic Methods

Method	Size Limitations	Comments
Rational	0-50 Acres	Method can be used for estimate peak flows and the design of small sub-division type storm sewer systems. Method shall not be used for storage design.
SCS/USGS HYDROS Model	0-25 Sq. Miles	Method can be used for estimating peak flows and hydrographs. Method can be used for the design of all drainage structures including storage facilities.
USGS Design	25 Acres to 25 Sq. Miles	Method can be used for estimating peak flows for all applications.
USGS	128 Acres to 25 Sq. Miles	Method can be used for estimating hydrographs for all design applications.
HEC-1* (U.S. Corps of Engineers Model)	>300 Acres	Method can be used for estimating peak flows and hydrographs. Method is recommended for hydrologic analysis of large drainage areas.

2.1.2 Design Frequency

Depending on the facility that is being designed or analyzed, different design frequencies will be required (see Section 2.3.1). In addition to the design frequency, the operations of all major drainage facilities (i.e. culverts, bridges) shall be checked using the 100-year frequency storm to ensure that there are no unexpected flood hazards. Thus, the operation of major drainage structures shall be checked for their design frequency and the 100-year storm.

All storage facilities shall be designed using the 2-, 5-, 10-, and 25-year storms with emergency facilities designed for the 100-year storm.

2.1.3 Symbols and Definitions

To provide consistency within this section as well as throughout this manual, the symbols presented in Table 2-2 will be used. These symbols were selected because of their wide use in hydrologic publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

2.2 Service Charge Credits and Exemptions Policy

a) The City of Loganville may give consideration to specific or unusual service requirements and general benefits accruing to or from properties as a result of providing their own stormwater management facilities.

b) The City of Loganville may reduce the service charge for certain properties if permanent detention facilities, designed, constructed, and maintained in accordance with the City of Loganville rules, regulations and standards, result in a reduction of actual contribution of runoff to the City of Loganville system. Sites with temporary facilities are not eligible for reductions in the service charge.

c) The City of Loganville may reduce the service charge for certain developments if the development can show evidence of increased water quality runoff from the site.

2.3 Design Frequency

2.3.1 Design Frequency

Cross Drainage: Cross drainage facilities transport storm runoff under roadways. Such drainage facilities shall be designed to accommodate a 25-year flood.
The design shall be such that the backwater caused by the structure (the headwater) does not:

- Increase the flood hazard significantly for adjacent property, or
- Overtop the roadway.

Thus the cross drainage shall be designed so that the roadway is not overtopped for all floods that are equal or less than the 25-year frequency event. Based on these design criteria, a design involving temporary roadway overtopping is acceptable practice for design frequencies, which exceed the 25-year flood. The final should be checked using the 100-year flood to be sure structures are not flooded or increased damage does not occur to the highway or adjacent property for this design event. Note: If roadway is used to impound floodwater is shall be analyzed structurally as a dam to ensure that failure will not occur for the 25-and 100-year flood events.

<u>Storm drains</u>: A storm drain shall be designed to accommodate a 25-year flood. The design shall be such that the storm runoff does not:

- Increase the flood hazard significantly for property,
- Encroach on to the street or highway so as to cause a significant hazard,
- Limit traffic, emerging vehicles, or pedestrian movements to an unreasonable extent.

Based on these design criteria, a design involving temporary street or road inundation is acceptable practice for flood events greater than the 25-year event but not for floods that are equal to or less than the 25-year event. Thus if a storm drainage system crosses under a roadway, then a 25-year flood must be routed through the system to show that the roadway will not be overtopped by this event. The excess storm runoff from events larger than the 25-year storm may be allowed to inundate the roadway or may be stored in areas other than on the roadway until the drainage system can accommodate the additional runoff.

<u>Inlets</u>: Inlets shall be designed for a 25-year flood event (see Section 3.1.3 in Chapter 3 – Storm Drainage Systems).

<u>Detention and retention storage facilities</u>: All storage facilities shall be designed to provide sufficient storage and release rates to accommodate the 2-, 5-10-, and 25-year design storm events. The design shall be such that the storm runoff does not:

- Increase the flood hazard significantly for adjacent, upstream, or downstream property as defined in the drainage ordinance, or
- Cause any safety hazards associated with the facility.

Emergency spillway facilities shall be provided to accommodate the 100-year storm.

The final design shall be checked to be sure that the downstream flood peaks for the storage discharge hydrographs have not increased for the 2-, 5-, 10-, and 25-year floods.

The storage facility must also accommodate the 100-year flood without causing damages to the structure or adjacent property.

2.3.2 Rainfall Intensity

The following rainfall intensities (Table 2-3) shall be used for all hydrologic analysis.

2.4 Rational Method

2.4.1 Introduction

When using the rational method some precautions should be considered.

- In determining the C value (land use) for the drainage area, hydrologic analysis should take into account future land use changes. Drainage facilities should be designed for future land use conditions as specified in the City of Loganville Land Use Plan.

▸ Since the rational method uses a composite C value for the entire drainage area, if the distribution of land uses within the drainage basin will affect the results of hydrologic analysis, then the basin should be divided into two or more subdrainage basins for analysis.

▸ The charts, graphs, and tables included in this section are given to assist the engineer in applying the rational method. The engineer should use good engineering judgment in applying these design aids and should make appropriate adjustments when specific site characteristics dictate that these adjustments are appropriate.

2.4.2 Equation

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed). The rational formula is expressed as follows:

Q=CIA

Where: Q = maximum rate of runoff (cfs)
C = runoff coefficient representing a ratio of runoff to rainfall
I = average rainfall intensity for a duration equal to the time of concentration (in./hr)
A = drainage area contributing to the design location (acres)

2.4.3 Time of Concentration

Use of the rational formula requires the time of concentration (tc) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I) from Table 2-3. The time of concentration consists of an overland flow time to the point where the runoff enters a defined drainage feature (i.e., open channel) plus the time of flow in a closed conduit or open channel to the design point.

Figure 2-1 can be used to estimate overland flow time. For each drainage area, the distance determined from the inlet to the most remote point in the tributary area. From a topographic map, the average slope is determined for the same distance. The runoff coefficient (C) is determined by the procedure described in the following subsections. Other methods and charts may be used to calculate overland flow time if approved by the City of Loganville.

To obtain the total time of concentration, the pipe or open channel flow time must be calculated and added to the inlet time. After first determining the average flow velocity in the pipe or channel, the travel time is obtained by dividing velocity into the pipe or channel length. Velocity can be estimated by using the nomograph shown on Figure 2-2. Note: Time of concentration cannot be less than five minutes.

Two common errors should be avoided when calculating time of concentration tc. First, in some cases runoff from a portion of the drainage area that is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Second, when designing a drainage system, the overland flow path is not necessarily the same before and after development and grading operations have been completed. Selecting overland path in excess of 100 feet in urban areas and 300 feet in rural areas should be done only after careful consideration.

2.4.4 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in in./hr for a duration equal to the time of concentration for a selected return period. Once a particular return has been selected for design and a time of concentration calculated for the drainage area, the rainfall intensity can be determined from Rainfall-Intensity-Duration data. Table 2-3 gives the data for The City of Loganville. Straight-line interpolation can be used to obtain rainfall intensity values for storm duration between the values presented on Table 2-3.

2.4.5 Runoff Coefficient

The runoff coefficient (C) is the variable of the rational method least susceptible to precise determination and requires judgment and understanding on the part of the design engineer. While engineering judgment will always be required in the selection of runoff coefficients, typical coefficients represent the integrated effects of many drainage basin parameters. Table 2-4 gives the recommended runoff coefficients for the Rational Method.

2.4.6 Composite Coefficients

It is often desirable to develop a composite runoff coefficient based on the percentage of different types of surfaces in the drainage areas. Composites can be made with the values from Table 2-4 by using percentages of different areas. Composites can be made with the values from Table 2-4 by using percentages of different land uses. In addition, more detailed composites can be made with coefficients for different surface types such as roofs, asphalt, and concrete streets, drives, and walks. The composite procedure can be applied to entire drainage areas or to typical “sample” blocks as a guide to the selection of reasonable values of the coefficient for an entire area.

2.5 Example Problem – Rational Method

2.5.1 Introduction

Following is an example problem which illustrates the application of the Rational Method to estimate peak discharges

2.5.2 Problem

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 25-year and 100-year return period.

2.5.3 Site Data

From a topographic map and field survey, the area of the drainage basin upstream from the point in question is found to be 18 acres. In addition the following data were measured:

Average overland slope =	2.0%
Length of overland flow =	50 ft
Length of main basin channel =	1300 ft
Slope of channel – 0.018 ft/ft =	1.8%

Roughness coefficient (n) of channel was estimated to be 0.090

2.5.4 Land Use and Soils

From existing land use maps, land use for the drainage basin was estimated to be:

Residential (single family)	80%
Graded – sandy soil, 3% slope	20%

From existing land use maps, the land use for the overland flow area at the head of the basin was estimated to be:

Lawn – sandy soil, 2% slope	100%

2.5.5 Overland Flow

A runoff coefficient (C) for the overland flow area is determined from Table 2-5 to be 0.15.

2.5.6 Time of Concentration

From Figure 2-1 with an overland flow length of 50 ft, slope of 2.0 percent and a C of 0.15, the overland flow time is 10 min. Channel flow velocity is determined from Figure 2-2 to be 3.5 ft/s (n = 0.090, R = 1.97 and S = .018). Therefore,

$$\text{Flow Time} = \frac{1300 \text{ feet}}{(3.5 \text{ ft/s})\ (60 \text{ s/min})} = 6.2 \text{ minutes}$$

$$\text{and } tc = 10 + 6.2 = 16.2 \text{ min (say 16 minutes)}$$

2.5.7 Rainfall Intensity

From Table 2-3 with duration equal to 16 min (values obtained by linear interpolation between values for 15 and 30 minutes)

I25 (25-year return period) = 5.9 in./hr
I100 (100-year return period) = 7.2 in./hr

2.5.8 Runoff Coefficient

A weighed runoff coefficient (C) for the total drainage area is determined in the following table by utilizing the values from Table 2-4

Land Use	(1) Percent of Total Land Area	(2) Runoff Coefficient	(3) Weighed Runooff Coefficent
Residential	.80	.50	.40
(single family)			
Graded area	.20	.30	.06
Total Weighted Runoff Coefficient			.46

* Column 3 equals column 1 multiplied by column 2.

2.5.9 Peak Runoff

From the rational method equation:

Q25 = CIA = 0.46 X 5.9 X 18 = 49 cfs
Q100 = CIA = 0.46 X 7.2 X 18 = 60 cfs

These are estimates of peak runoff for a 25-year and 100-year design storm for the given basin.

2.6 SCS Unit Hydrograph

2.6.1 Introduction

The Soil Conservation Service (SCS) hydrologic method required basic data similar to the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. Details of the methodology can be found in the SCS National Engineering Handbook, Section 4.

The SCS method includes the following basic steps:

1. Determination of curve numbers that represent different land uses within the drainage area.

2. Calculation of time of concentration to the study point.

3. Using the Type II rainfall distribution, total and excess rainfall amounts are determined.

4. Using the hydrograph approach, triangular, and composite hydrographs are developed for the drainage area.

2.6.2 Equations and Concepts

The following discussion outlines the equations and basic concepts used.

Drainage Area – The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, and route flows to points of interest.

Rainfall – The SCS method applicable to the City of Loganville is based on a storm event, which has a Type II time distribution. Figure 2-3 shows this distribution. To use this distribution it is necessary for the user to obtain the 24-hour rainfall volume (P24 in Figure 2-3) from Table 2-3 for the frequency of the design storm. This volume is then distributed according to Figure 2-3.

Rainfall-Runoff Equation – A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The following SCS runoff equation is used to estimate direct runoff from 24-hour or 1-day storm rainfall. The equation is:

$$\mathbf{Q = (P - Ia)2 / (P - Ia) + S \qquad (2.3)}$$

Where: Q = accumulated direct runoff
P = accumulated rainfall (potential maximum runoff)
Ia = initial abstraction including surface storage, interception, and infiltration prior to runoff
S = potential maximum soil retention

The empirical relationship used in the SCS method for estimating Ia is:

$$\mathbf{Ia = 0.2S \qquad (2.4)}$$

Substituting 0.2S for Ia in equation 2.3 the equation becomes:

$$Q = (P-0.2S)2 / (P + 0.8S) \qquad (2.5)$$

Where: $S = 1000/CN – 10$ and
CN = SCS Curve Number

Figure 2-4 shows a graphical solution of this equation. For example, 4.1 inches of direct runoff would result if 5.8 inches of rainfall occurs on a watershed with a curve number of 85.

2.6.3 Runoff Factor

The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land scope. The SCS method uses a combination of soil conditions and land-uses (ground cover) to assign a runoff factor to an area. These runoff factors, called curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential. Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Based on infiltration rates, SCS has divided soils into four hydrologic soil groups.

Group A Soils having low runoff potential due to high infiltration rates. These soils consist primarily of deep, well-drained sands and gravels.

Group B Soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately-deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures.

Group C Soils having a moderately high runoff potential due to slope infiltration rate. These soils consist primarily of soils in which a layer exist near the surface that impede the downward movement of water of soils with moderately-fine to fine texture.

Group D Soils have a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

The soils within the City of Loganville are typically of the Appling-Louisburg-Cecil association: well drained to somewhat excessively drained soils on uplands; gently sloping ridgetops and fairly steep side slopes. Chenacla and Wehadkee soils can be found on nearly level floodplains like Big Flat Creek and Little Haynes Creek.

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface soils, appropriate changes should be made in the soil group selected. Also, runoff curve numbers vary with the antecedent soil moisture conditions. Average antecedent soil moisture conditions (AMC II) are recommended for all hydrologic analysis.

Table 2-5 gives recommended curve number values for a range of different land uses.

2.6.4 Urban Modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2-5 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over a pervious area and then into a drainage system. It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. For a discussion of impervious areas and their effect on curve number values see Section 2.7.

2.6.5 Travel Time Estimation

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$\mathbf{Tt = L/(3600V)} \qquad \mathbf{(2.6)}$$

Where: Tt = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours

2.6.6 Sheet Flow

Sheet flow can be calculated using the following equations:

$$\mathbf{Tt = [0.42\ (nL)0.8\ /\ (P2)0.5\ (S)0.4]} \qquad \mathbf{(2.7)}$$

Where: Tt = travel time (min.)
N = Manning roughness coefficient (see Table 2-6
L = flow length (ft)
P2 = 2-year, 24-hour rainfall = 3.6 in, and
S = slope of hydraulic grade line = (land slope ft/ft)

Substituting the constant rainfall amount the travel time equation becomes:

$$\mathbf{Tt = [0.221\ (nL)0.8]\ /\ (S)0.4} \qquad \mathbf{(2.8)}$$

Thus the final equations for paved and unpaved areas are:

Paved:

$$Tt = 0.0060[(L)^{0.8} / (S)^{0.4}] \qquad (2.9)$$
$$V = 2.68\,(S)^{0.4}\,(L)^{0.2} \qquad (2.10)$$

Unpaved:

$$Tt = 0.0073[(L)^{0.8} / (S)^{0.4}] \qquad (2.11)$$
$$V = 0.22\,(S)^{0.4}\,(L)^{0.2} \qquad (2.12)$$

Where: V = velocity (ft/s)

2.5.6.1Shallow Concentrated Flow

After a maximum of 300 feet (100 feet in urban areas), sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from Figure 2-7, in which average velocity is a function of watercourse slope and type of channel.

Average velocities for estimating travel time for shallow concentrated flow can be computed from using Figure 2-7, or the following equations. These equations can also be used for slopes less than 0.005 ft/ft.

Unpaved:

$$V = 16.1345(S)^{0.5} \qquad (2.13)$$

Paved:

$$V = 20.3282(S)^{0.5} \qquad (2.14)$$

These two equations are based on the solution of the Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, ft) for unpaved areas, n is 0.05 and r is 0.4; for paved areas, n is 0.025 and r is 0.2.
After determining average velocity using Figure 2-7 or equations 2.13 or 2.14, use equation 2.6 to estimate travel time for the shallow concentrated flow segment.

2.6.6.1 Open Channels

Open channels are assumed to where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation of water surface information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation. Manning's equation is:

$$V = [1.49(r)^{2/3}\,(s)^{1/2}] / n \qquad (2.15)$$

Where: V = average velocity (ft/s)
r = hydraulic radius (ft) and is equal to a/pw

a = cross sectional flow area (ft2)
pw = wetted perimeter (ft)
s = slope of the hydraulic grade line (ft/ft)
n = Manning's roughness coefficient for open channel flow

After average velocity is computed using the equation 2.15, Tt for the channel segment can be estimated using equation 2.6.

Velocity in channels should be calculated from the Manning's equation. Cross sections from all channels that have been field checked should be used in the calculations. This is particularly true of areas below dams or other flow control structures.

2.6.6.2 Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet (100 feet in urban areas).
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate Tc.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. Detailed storage routing procedures should be used to determine the outflow through the culvert.

2.6.6.3 Triangular Hydrograph Equation

The triangular hydrograph is a practical representation of excess runoff with only one rise, one peak, and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the process of estimating discharge rates, and produces results that are sufficiently accurate for most drainage facility designs. The SCS developed the following equation to estimate the peak rate of discharge for an increment of runoff:

qp = (484 A (q) / (d/2 + TL) (2.16)

Where: qp = peak rate of discharge (cfs)
A = area (mi2)
q = storm runoff volume during time interval (in.) (obtained from equation 2.13)
d = rainfall time increment
TL = watershed lag time (TL = 0.6 Tc) (hr)
Tp = time to peak (d/2 + TL) (hr)
Tb = time of base (2.67 Tp) (hr)

2.7 Impervious Area Calculations

2.7.1 Urban Modifications

Several factors, such as the percentage of impervious area and the means of conveying runoff from impervious areas to the drainage system, should be considered in computing CN for urban areas. For example, do the impervious areas connect directly to the drainage system, or do they outlet onto lawns or other pervious areas where infiltration can occur?

The curve number values given in Table 2-7 are based on directly connected impervious area. An impervious area is considered directly connected if runoff from it flows directly into the drainage system. It is also considered directly connected if runoff from it occurs as concentrated shallow flow that runs over pervious areas and then into a drainage system.

It is possible that curve number values from urban areas could be reduced by not directly connecting impervious surfaces to the drainage system. The following discussion will give some guidance for adjusting curve numbers for different types of impervious areas.
Connected Impervious Areas

Urban CN's given in Table 2-7 were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that:

(a) pervious urban areas are equivalent to pasture in good hydrologic condition, and
(b) impervious areas have a CN of 98 and are directly connected to the drainage system.

If all of the impervious area is directly connected to the drainage system, but the impervious area percentages or the pervious land use assumptions in Table 2-5 are not applicable, use Figure 2-5 to compute a composite CN. For example, Table 2-5 gives a CN of 70 for a ½-acre lot in hydrologic soil group B, with an assumed impervious area of 25 percent. However, if the lot has 20 percent impervious area and a pervious area CN of 61, the composite CN obtained from Figure 2-5 is 68. The CN difference between 70 and 68 reflects the difference in percent impervious area.

Unconnected Impervious Areas

Runoff from these areas is spread over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system, (1) use Figure 2-6 if total impervious area is less than 30 percent or (2) use Figure 2-5 if the total impervious area is equal to or greater than 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.

When impervious area is less than 30 percent, obtain the composite CN by entering the right half of Figure 2-6 with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a ½ acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from Figure 2-6 is 66. If all of the impervious areas are connected, the resulting CN (from Figure 2-5) would be 68.

2.8 Composite Curve Numbers

When a drainage area has more than one land use, a composite curve number can be calculated and used in the analysis. It should be noted that when composite curve numbers are used, the analysis does not take into account the location of the specific land uses but sees the drainage area as a uniform land use represented by the composite curve number.

Composite curve numbers for a drainage area can be calculated by entering the required data into a table such as the one presented in Table 2-6

Table 2-6Composite Curve Number Calculations

(1) Land Use	(2) Curve Number	(3) Area	(4) % of Total Area	(5) Composite Curve Number (Col. 2 x Col. 4)

The composite curve number for the total drainage area is then the sum of the composite curve numbers from column 5.

The different land uses within the same basin should represent a uniform hydrologic group represented by a single curve number. Any number of land uses can be included but if their spatial distribution is important to the hydrologic analysis then sub-basins should be developed and separate hydrographs developed and routed to the study point.

2.9 Simplified SCS Method

2.9.1 Overview

The following SCS procedures were taken from the SCS Technical Release 55 (TR-55), which presents simplified procedures to calculate storm runoff volume, peak rate discharges and hydrographs. These procedures are applicable to small drainage areas and include provisions to account for urbanization. The following procedures outline the use of the SCS-TR-55 method.

2.9.2 Peak Discharge

The SCS peak discharge method is applicable for estimating the peak runoff rate from watersheds with homogeneous land uses. The following method is based on the results of computer analyses performed using TR-20, "Computer Program Project Formulation – Hydrology (SCS 1983).

The peak discharge equation is:

$$Qp = quAQFp \qquad (2.17)$$

Where: Qp = peak discharge (cfs)
qu = unit peak discharge (cfs/mi2/in)
A = drainage area (mi2)
Q = runoff (in)
Fp = pond and swamp adjustment factor

The input requirements for this method are as follows:

1. Tc – hours
2. Drainage area – mi2
3. Type II rainfall distribution
4. 24-hour design rainfall
5. CN Value
6. Pond and Swamp adjustment factor (If pond and swamp areas are spread throughout the watershed and are not considered in the Tc computation, an adjustment is needed.)

2.9.3 Computations

Computations for the peak discharge method proceed as follows:

1. The 24-hour rainfall depth is determined from the following table for the selected return frequency.

Frequency	24-hour Rainfall
2-year	3.8 inch
5-year	4.95 inch
10-year	5.8 inch
25-year	6.7 inch
50-year	7.7 inch
100-year	8.0 inch

Source: National Weather Service, Tech. Paper 40, "Rainfall Frequency Atlas of the U.S.", May 1961.

2. The runoff curve number, CN, is estimated from Table 2-5 and direct runoff, Q, is calculated using equation 2.17.

3. The CN value is used to determine the initial abstraction, Ia, from Table 2-7 and the ratio Ia/P is then computed. (P=accumulated rainfall or potential maximum runoff.)

4. The watershed time of concentration is computed using the procedures in Section 2.10.6 and is used with the ratio Ia/P to obtain the unit peak discharge, qu, from Figure 2-8. If the ratio Ia/P lies outside the range shown in Figure 2-8, either the limiting values or another peak discharge method should be used.

5. The pond and swamp adjustment factor, Fp, is estimated from below:

Pond & Swamp Areas (%)	Fp
0	1.00
0.2	.97
1.0	.87
3.0	.75
5.0	.72

6. The peak runoff rate is computed using equation 2.17.

2.9.4 Limitations

The accuracy of the peak discharge method is subject to specific limitations, including the following.

2. The watershed must be hydrologically homogeneous and describable by a single CN value.

3. The watershed may have only one main stream, or if more than one, the individual branches must have nearly equal time of concentrations.

4. Hydrologic routing cannot be considered.

5. The pond and swamp adjustment factor, Fp, applies only to areas located away from the main flow path.

6. Accuracy is reduced if the ratio Ia/P is outside the range given in Figure 2-8.

7. The weighted CN value must be greater than or equal to 4-and less than or equal to 98.

8. The same procedure should be used to estimate pre-and post-development time of concentration when computing pre-and post-development peak discharge.

9. The watershed time of concentration must be between 0.1 and 10 hours.

2.9.5 Example Problem

Compute the 25-year peak discharge for a 50-acre wooded watershed, which will be developed as follows:

1. Forest land – poor cover (hydrological soil Group B) =10 ac.
2. Forest land – poor cover (hydrological soil Group C) =10 ac.
3. Townhouse residential (hydrological soil Group B) =20 ac.
4. Industrial development (hydrological soil Group C) =10 ac.

Other data include: percentage of pond and swamp area = 0.

Computations

1. Calculate rainfall excess:

- The 25-year, 24-hour rainfall is 6.55 inches.
- Composite weighted runoff coefficient is:

Dev. #	Area	% Total	CN	Composite
1	10 ac.	.20	55	11.0
2	10 ac.	.20	70	14.0
3	20 ac.	.40	85	34.0
4	10 ac.	.20	91	18.2
TOTAL	50 ac.	1.00		77.2 use 77

* From Figure 2-4, Q = 4.0 inches

2. Calculate time of concentration

- The hydrologic flow path for this watershed = 2,000 feet

Segment	Type of Flow	Length	Slope (%)
1	Overland n = 0.45	70 ft.	2.0%
2	Shallow Channel	750 ft.	1.7%
3	Main Channel	1100 ft.	0.20%

* For the main channel, n = 0.025 (from Manning's n book), width = 10 feet, depth = 2 feet, rectangular
channel.

- Segment 1 – Travel time from equation 2.7 with P2 = 3.73 in.

Tt = [0.42(0.45 X 60)0.8] / [(3.73)0.5 (0.035)0.4]
Tt = 11.6 minutes

- Segment 2 – Travel time from Figure 2-7 and equation 2.6

V = 2.6 ft/sec (from Figure 2-7 or equation 2.14)
Tt = 750 / 60 (2.6) = 4.8 minutes

- Segment 3 – Using equation 2.15

V = (1.49 / 0.025) (1.43)0.67 (.002).5 = 3.4 ft/sec
Tt = 1100 / 60 (3.4) = 5.4 minutes

- Tc = 11.8 + 4.6 + 5.4 = 21.8 minutes (.36 hours)

3. Calculate Ia/P for CN = 77 (Table 2-5), Ia = 0.597 (Table 2-7)

   Ia/P = (0.597 / 6.55) = 0.091

   (Note: Use Ia/P = 0.10 to facilitate use of SCS Unit Discharge Hydrographs. Straight-line interpolation could also be used.)
4. Estimate unit discharge qu from Figure 2-8 = 620 cfs
5. Calculate peak discharge with Fp = 1 using equation 2.17

   Q25 = 620 (50/640) (4.0) (1) = 194 cfs.

2.9.6 Hydrograph Generation

In addition to estimating the peak discharge, the SCS method can be used to estimate the entire hydrograph. The Soil Conservation Service has developed a Tabular Hydrograph procedure, which can be used to generate the hydrograph for small drainage areas. The Tabular Hydrograph procedure uses unit discharge hydrographs, which have been generated for a series of time of concentrations (see example on next page). This method is presented in Chapter 5, Tabular Hydrograph Method "Appendix A-1, Estimating Runoff from Urban Areas" from the Manual for Erosion and Sediment Control in Georgia (210-VI-TR-55, Second Ed., June 1986).

2.9.7 Composite Hydrograph

The procedures given in this chapter are for generation of a hydrograph from a homogeneous developed drainage area. For drainage areas which are not homogeneous where hydrographs need to be generated from sub-areas and then routed and combined at a point downstream, the engineer is referred to either the HYDROS model or the procedures outlined by the SCS in the 1986 version of TR-55 available from the National Technical Information Service in Springfield, Virginia 22161. The catalog number for TR-55, "Urban Hydrology for Small Watersheds," is PB87-101580.

2.9.8 Example Problem

For the example problem in 2.9.5, calculate the entire hydrograph from the 50-acre development.

Using the Tabular Hydrograph charts presented in Appendix A-1 of the Manual for Erosion and Sediment Control in Georgia with a time of concentration of 0.37 hours and Ia/P = 0.10 the following hydrograph can be generated (using straight line interpolation between time of concentration of 0.3 and 0.4 hours).

2.10 U.S. Geological Survey Peak Flow and Hydrograph Method

2.10.1 Introduction

For the past 20 years the U.S. Geological Survey has been collecting rain and streamflow data at various sites within the Atlanta Metropolitan Area and throughout the State of Georgia. The data from these efforts have been used to calibrate a U.S. Geological Survey rainfall-runoff model for use within the Atlanta Area. The U.S. Geological Survey Model was then used to develop peak discharge regression equations for the 2-, 5-, 10-, 25-, 50-, and 100-year floods. In addition, the USGS used the statewide database to develop a dimensionless hydrograph, which can be used to simulate flood hydrographs from rural and urban streams within The City of Loganville.

2.10.2 Peak Discharge Equations

For a complete description of the USGS regression equations presented below, consult the USGS publication "Flood-Frequency Relations for Urban Streams in Georgia, Water Resources Investigation Report 88-4085". Following are the USGS regression equations for use in The City of Loganville.

Frequency	Equation
2-year	Q2=145A.70TIA.0.31
5-year	Q5=258A.69TIA.26
10-year	Q10=351A.70TIA.21
25-year	Q25=452A.70TIA.20
50-year	Q50=548A.70TIA.18
100-year	Q100=644A.70TIA.17
	A = drainage area, mi2
	TIA = total impervious area, %

2.10.3 Limitations

Following are the limitations of the variables within the peak discharge equations. These equations should not be used on drainage areas that have physical characteristics outside the limits listed below.

Physical Characteristics	Minimum	Maximum	Units
A – Drainage Area	0.04	25	mi2
TIA – Total Impervious Area	1.00	62	percent

2.10.4 Hydrographs

The USGS has developed a dimensionless hydrograph for Georgia streams having drainage areas of less than 500 mi2. This dimensionless hydrograph can be used to simulate flood hydrographs for rural and urban streams within The City of Loganville. For a complete description of the USGS dimensionless hydrograph consult the USGS publication "Simulation of Flood Hydrographs for Georgia Streams", Water-Resources Investigation Report 86-4004, Following are the time and discharge ratios for the dimensionless hydrograph for The City of Loganville.

2.10.5 Limitations

Following are the limitations of the variables within the lag time equation (2.18). The lag time equation should not be used for drainage areas that have physical characteristics outside the limits listed below

Physical Characteristics	Minimum	Maximum	Units
A – Drainage Area	0.2	25	mi2
S – Main Channel Slope	13	175	feet/mi
IA-Total Impervious Area	14	50	percent

2.10.6 Rural Basins

The USGS has recently revised the equation for estimating peak discharges for rural basins. For a complete discussion of the development of these equations consult the USGS publication "Techniques for Estimating Magnitude and Frequency of Floods in Rural Basins of Georgia", Water-Resources Investigations Report 95-4017.

For these rural equations, the USGS Region 2 will affect the use of the equations within Walton County. Region 2 includes all those drainage areas that are should of the major drainage divide through Atlanta and eventually flow into the South River, Flint River, etc. Walton County is located in Region 2. Following are the equations used to calculate peak discharges for rural basins in Walton County.

Frequency	Equation Region 2
RQ2	182A0.622
RQ5	311A0.616
RQ10	411A0.613
RQ25	552A0.610
RQ50	669A0.607

RQ_{100}	$794A^{0.605}$

A = Drainage Area in mi2

2.10.7 Limitations

Following are the limitations associated with the rural basis equations given above.

Physical Characteristics	Minimum	Maximum	Units
A – Drainage Area	0.17	730	mi2

2.10.8 Example Problem

For the 10-year flood, calculate the peak discharge for rural and developed conditions for the following drainage area.

Drainage Area = 175 acres = 0.273 mi2
Drainage Area is located in Region 1
Total Impervious Area (TIA) = 32%

<u>10-year Rural Peak Discharge</u>

$RQ_{10} = 411A^{0.613} = 411(.273)^{0.613} = 185$ cfs

<u>10-year Developed Peak Flow</u>

$Q_{10} = 351A^{0.70}TIA^{0.21}$

$Q_{10} = 351\ (.273)^{.70}\ (32)^{.21} = 293$ cfs

2.10.9 References

Federal Highway Administration. 1991. HYDRAIN Documentation.

U.S. Department of Agriculture, Soil Conservation Service, Engineering Division. 1986. Urban Hydrology for Small Watersheds. Technical Release 55 (TR-55).

U.S. Department of Transportation, Federal Highway Administration. 1984. Hydrology. Hydraulic Engineering Circular No. 19.

Overton, D.E. and M.E. Meadows. 1976. Stormwater Modeling. Academic Press. New York, N.Y. pp.58-88.

Water Resources Council Bulletin 17B. 1981. Guidelines for Determining Flood Flow Frequency.

Wright-McLaughlin Engineers. 1969. Urban Storm Drainage Criteria Manual. Vol. I and II. Prepared for the Denver Regional Council of Governments, Denver, Colorado.

3.1 Overview

3.1.1 Introduction

In this chapter, guidelines are given for evaluating roadway features and design criteria as they relate to gutter and inlet hydraulics and storm drain design. Procedures for performing gutter flow calculations are based on a modification of Manning's Equation. Inlet capacity calculations for grate and combination inlets are based on information contained in HEC-12 (USDOT, FHWA, 1984). Storm drain design is based on the use of the rational formula.

3.1.2 Inlet Definition

There are three stormwater inlet categories:

- curb opening inlets
- grated inlets
- combination inlets

I
n addition, inlets may be classified as being on a continuous grade or in a sump. The term "continuous grade" refers to an inlet located on the street with a continuous slope past the inlet with water entering from one direction. The "sump" condition exists when the inlet is located at a low point and water enters from both directions.

3.1.3 Criteria

<u>Design Frequency</u>

1. The 25-year design storm shall be used for conveyance systems and drainage designs.
2. The 50-year design storm shall be used for conveyance systems under arterial roads.

<u>Spread Limits</u>

Catch basins shall be spaced so that the spread in the street for the 25-year design flow shall not exceed the following, as measured from the face of the curb:

1. 8 feet if the street is classified as a Collector or Arterial (for 2 lane streets spread may extend to one-half of the travel lane, for 4 lane streets spread may extend across one travel lane);
2. 16 feet at any given section, but in no case greater than 10 feet on one side of the street, if the street is classified as a Local or Sub-Collector Street.

3.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in storm drainage publications. In some cases the same symbol is used in existing publications for more than one definition. Where this occurs in this chapter, the symbol will be defined where it occurs in the text or equations.

3.3 Gutter Flow Calculations

3.3.1 Formula

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = [0.56 / n]\ S_x^{5/3}\ S^{1/2}\ T^{8/3} \qquad (3.1)$$

Where: Q= gutter flow rate, cfs
n = Manning's roughness coefficient
S_x =pavement cross slope, ft/ ft
S = longitudinal slope, ft/ ft
T = width of flow or spread, ft

3.3.2 Nomograph

A nomograph for solving Equation 3.1 is presented Figure 3-1. Manning's n values for various pavement surfaces are presented in the table below.

3.3.3 Manning's n Table

Table 3-2 Manning's n Values For Street And Pavement Gutters

Type of Gutter or Pavement	Range of Manning's n
Concrete gutter, troweled finish	0.012
Asphalt pavement:	
Smooth texture	0.013
Rough texture	0.016

Concrete gutter with asphalt pavement	
Smooth	0.013
Rough	0.015
Concrete pavement	
Float finish	0.014
Broom finish	0.016
For gutters with small slopes, where sediment may accumulate, increase above values of n by	0.002

Note: Estimates are by the Federal Highway Administration
Reference: USDOT, FHWA, HDS-3(1961)

3.3.4 Uniform Cross Slope

The nomograph presented in Figure 3-1 is used with the following procedures to find gutter capacity for uniform cross slopes:

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope (S), cross slope (Sx), gutter flow (Q), and Manning's n.

2. Draw a line between the S and Sx scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate gutter flow value on the capacity scale. If Manning's n is 0.016, use Q from Step 1; if not, use the product of Q and n.

4. Read the value of the spread (T) at the intersection of the line from Step 3 and the spread scale.

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including longitudinal slope (S), cross slope (Sx), spread (T), and Manning's n.

2. Draw a line between the S and Sx scales and note where it intersects the turning line.

3. Draw a line between the intersection point from Step 2 and the appropriate value on the T scale. Read the value of Q or Qn from the intersection of that line on the capacity scale.

4. For Manning's n values of 0. 016, the gutter capacity (Q) from Step 3 is selected. For other Manning's n values, the gutter capacity times n (Qn) is selected from Step 3 and divided by the appropriate n value to give the gutter capacity.

3.3.5 Composite Gutter Sections

Figure 3-2 in combination with Figure 3-1 can be used to find the flow in a gutter with width (W) less than the total spread (T). Such calculations are generally used for evaluating composite gutter sections or frontal flow for grate inlets.

Figure 3-3 provides a direct solution of gutter flow in a composite gutter section. The flow rate at a given spread or the spread at a known flow rate can be found from this figure. Figure 3-3 involves a complex graphical solution of the equation for flow in a composite gutter section. Typical of graphical solutions, extreme care in using the figure is necessary to obtain accurate results.

Condition 1: Find spread, given gutter flow.

1. Determine input parameters, including longitudinal slope (S), cross slope (Sx), depressed section slope (Sw), depressed section width (W), Manning's n, gutter flow (Q), and a trial value of gutter capacity above the depressed section (Qs).

2. Calculate the gutter flow in W (Qw), using the equation:

   Qw = Q – Qs **(3.2)**

3. Calculate the ratios Qw/Q or Eo and Sw /Sx and use Figure 3-2 to find an appropriate value of W/ T.

4. Calculate the spread (T) by dividing the depressed section width (W) by the value of W/ T from Step 3.

5. Find the spread above the depressed section (Ts) by subtracting W from the value of T obtained in Step 4.

6. Use the value of Ts from Step 5 along with Manning's n, S, and Sx to find the actual value of Qs from Figure 3-1.

7. Compare the value of Qs from Step 6 to the trial value from Step 1. If values are not comparable, select a new value of Qs and return to Step 1.

Condition 2: Find gutter flow, given spread.

1. Determine input parameters, including spread (T), spread above the depressed section (Ts), cross slope (Sx), longitudinal slope (S), depressed section slope (Sw), depressed section width (W), Manning's n, and depth of gutter flow (d).

2. Use Figure 3-1 to determine the capacity of the gutter section above the depressed section (Qs). Use the procedure for uniform cross slopes, substituting Ts for T.

3. Calculate the ratios W/ T and Sw/Sx, and, from Figure 3-2, find the appropriate value of Eo (the ratio of Qw/Q).

4. Calculate the total gutter flow using the equation:

$$Q = Q_s / (1 - E_o) \qquad \textbf{(3.3)}$$

Where: Q = gutter flow rate, cfs
Q_s = flow capacity of the gutter section above the depressed section,cfs
E_o = ratio of frontal flow to total gutter flow (Q_w/Q)

5. Calculate the gutter flow in width (W), using Equation 3.2.

3.3.6 Examples

Example 1

Given: $T = 8$ ft $S_x = 0.025$ ft/ ft
$n = 0.015$
$S = 0.01$ ft/ ft

Find: (1) Flow in gutter at design spread

(2) Flow in width ($W = 2$ ft) adjacent to the curb

Solution: (1) From Figure 3-1, $Qn = 0.03$

$Q = Qn/ n = 0.03/ 0.015 = 2.0$ cfs

(2) $T = 8 - 2 = 6$ ft

$(Qn)_2 = 0.014$ (Figure 3-1) (flow in 6 ft width outside of width W)

$Q = 0.014/ 0.015 = 0.9$ cfs

$Q_w = 2.0 - 0.9 = 1.1$ cfs

Flow in the first 2 ft adjacent to the curb is 1.1 cfs and 0.9 cfs in the remainder of the gutter.

Example 2

Given: $T = 6$ ft
$S_w = 0.0833$ ft/ ft
$T_s = 6 - 1.5 = 4.5$ ft
$W = 1.5$ ft
$S_x = 0.03$ ft/ ft $n = 0.014$ $S = 0.04$ ft/ ft

Find: Flow in the composite gutter

Solution: (1) Use Figure 3-1 to find the gutter section capacity above the depressed section.
$Q_sn = 0.038$
$Q_s = 0.038/ 0.014 = 2.7$ cfs

(2) Calculate $W/ T = 1.5/ 6 = 0.25$ and
$S_w / S_x = 0.0833/ 0.03 = 2.78$
Use Figure 3-2 to find $E_o = 0.64$

(3) Calculate the gutter flow using Equation 3.3
$Q = 2.7/(1 - 0.64) = 7.5$ cfs

(4) Calculate the gutter flow in width, W, using Equation 3.2
$Q_w = 7.5 - 2.7 = 4.8$ cfs

3.4 Grate Inlets Design

3.4.1 Types

Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or direct outlet to culverts. Grate inlets subject to traffic should be bicycle safe and be load bearing adequate. Appropriate frames should be provided.

Inlets used for the drainage of highway surfaces can be divided into three major classes.

1. Grate Inlets -These inlets include grate inlets consisting of an opening in the gutter covered by one or more grates, and slotted inlets consisting of a pipe cut along the longitudinal axis with a grate or spacer bars to form slot openings.

2. Curb-Opening Inlets -These inlets are vertical openings in the curb covered by a top slab.

3. Combination Inlets -These inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate.

In addition, where significant ponding can occur, in sag vertical curves in depressed sections, it is good engineering practice to place flanking inlets on each side of the inlet at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the level point and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded.

The design of grate inlets will be discussed in this section, curb inlet design in Section 3.5 and combination inlets in Section 3.6.

3.4.2 Grate Inlets On Grade

The capacity of an inlet depends upon its geometry and the cross slope, longitudinal slope, total gutter flow, depth of flow and pavement roughness. The depth of water next to the curb is the major factor in the interception capacity of both gutter inlets and curb opening inlets. At low velocities, all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. On steep slopes, only a portion of the frontal flow will be intercepted if the velocity is high or the grate is short and splash-over occurs. For grates less than 2 feet long, intercepted flow is small.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved vane grate and the tilt bar grate are recommended for both their hydraulic capacity and bicycle safety features. They also handle debris better than other grate inlets but the vanes of the grate must be turned in the proper direction. Where debris is a problem, consideration should be given to debris handling efficiency rankings of grate inlets from laboratory tests in which an attempt was make to qualitatively simulate field conditions. Table 3-3 presents the results of debris handling efficiencies of several grates.

The ratio of frontal flow to total gutter flow, Eo , for straight cross slope is expressed by the following equation:

$$\mathbf{Eo = Qw/Q = 1 - (1 - W/T)^{2.67}} \quad \textbf{(3.4)}$$

Where: Q = total gutter flow, cfs
Qw= flow in width W, cfs
W = width of depressed gutter or grate, ft
T = total spread of water in the gutter, ft

Figure 3-2 provides a graphical solution of Eo for either depressed gutter sections or straight cross slopes. The ratio of side flow, Qs, to total gutter flow is:

$$\mathbf{Qs/Q = 1 - Qw/Q = 1 - Eo} \quad \textbf{(3.5)}$$

The ratio of frontal flow intercepted to total frontal flow, Rf , is expressed by the following equation:

$$\mathbf{Rf = 1 - 0.09\,(V - V0)} \quad \textbf{(3.6)}$$

Where: V = velocity of flow in the gutter, ft/ s (using Q from Figure 3-1)
Vo = gutter velocity where splash-over first occurs, ft/ s (from Figure 3-4)

This ratio is equivalent to frontal flow interception efficiency. Figure 3-4 provides a solution of equation 3.6 which takes into account grate length, bar configuration and gutter velocity at which splash-over occurs. The gutter velocity needed to use Figure 3-4 is total gutter flow divided by the area of flow. The ratio of side flow intercepted to total side flow, Rs , or side flow interception efficiency, is expressed by:

$$\mathbf{Rs = 1 / [1 + (0.15V^{1.8}/Sx\ L^{2.3})]} \quad \textbf{(3.7)}$$

Where: L = length of the grate, ft

Figure 3-5 provides a solution to equation 3.7. The efficiency, E, of a grate is expressed as:

$$\mathbf{E = Rf\ Eo + Rs\ (1 - Eo)} \quad \textbf{(3.8)}$$

The interception capacity of a grate inlet on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$\mathbf{Qi = EQ = Q[\,Rf\ Eo + Rs\ (1 - Eo)]} \quad \textbf{(3.9)}$$

The following example illustrates the use of this procedure.

Given: W = 2 ft T = 8 ft

S_x= 0. 025 ft/ ft S = 0. 01 ft/ ft
E_o = 0.69 Q = 3.0 cfs
V = 3.1 ft/ s Gutter depression = 2 in
Find: Interception capacity of:
(1) a curved vane grate, and
(2) a reticulate grate 2-ft long and 2-ft wide

Solution:
From Figure 3-4 for Curved Vane Grate, R_f = 1.0
From Figure 3-4 for Reticulate Grate, R_f = 1.0
From Figure 3-5 R_s = 0.1 for both grates.
From Equation 3.9:
$Q_i = 3.0 [1.0 \times 0.69 + 0.1(1-0.69)] = 2.2$ cfs
For this example the interception capacity of a curved vane grate is the same as that for a reticulate grate for the sited conditions.

3.4.3 Grate Inlets In Sag
A grate inlet in a sag operates as a weir up to a certain depth dependent on the bar configuration and size of the grate and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0. 4 foot above the top of grate and when depth of water exceeds about 1.4 feet, the grate begins to operate as an orifice. Between depths of about 0.4 foot and about 1.4 feet, a transition from weir to orifice flow occurs.
The capacity of grate inlets operating as a weir is:
$Q_i = CPd^{1.5}$ (3.10)
Where: P = perimeter of grate excluding bar widths and the side against the curb, ft
C = 3.0
d = depth of water above grate, ft
and as an orifice is:
$Q_i = CA(2gd)^{0.5}$ (3.11)
Where: C = 0.67 orifice coefficient
A = clear opening area of the grate, ft^2
g = 32.2 ft/ s^2
Figure 3-6 is a plot of equations 3.10 and 3.11 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. The following example illustrates the use of this figure.

Given: A symmetrical sag vertical curve with equal bypass from inlets upgrade of the low point; allow for 50% clogging of the grate.
Q_b = 3. 6 cfs Q = 8 cfs, 25-year storm
T = 10 ft, design S_x= 0.05 ft/ ft
d = TS_x = 0.5 ft
Find: Grate size for design Q. Check spread at S = 0.003 on approaches to the low point.
Solution: From Figure 3-6, a grate must have a perimeter of 8 ft to intercept 8 cfs at a depth of 0.5 ft.

Some assumptions must be made regarding the nature of the clogging in order to compute the capacity of a partially clogged grate. If the area of a grate is 50 percent covered by debris so that the debris-covered portion does not contribute to interception, the effective perimeter will be reduced by a lesser amount than 50 percent. For example if a 2-ft x 4-ft grate is clogged so that the effective width is 1-ft, then the perimeter, P = 1 + 4 + 1 = 6 ft, rather than 8 ft, the total perimeter, or 4 ft, half of the total perimeter. The area of the opening would be reduced by 50 percent and the perimeter by 25 percent.

Therefore, assuming 50 percent clogging along the length of the grate, a 4 x 4, a 2 x 6, or a 3 x 5 grate would meet requirements of an 8-ft perimeter 50 percent clogged.

Assuming that the installation chosen to meet design conditions is a double 2 x 3 ft grate, for 50 percent clogged conditions: P = 1 + 6 + 1 = 8 ft

For 25-year flow: d = 0.5 ft (from Figure 3-6)

The American Society of State Highway and Transportation Officials (AASHTO) geometric policy recommends a gradient of 0.3 percent within 50 ft of the level point in a sag vertical curve.

Check T at S = 0.003 for the design and check flow:

Q = 3.6 cfs, T = 8.2 ft (25-year storm) (Figure 3-1)

Thus a double 2 x 3-ft grate 50 percent clogged is adequate to intercept the design flow at a spread which does not exceed design spread and spread on the approaches to the low point will not exceed design spread. However, the tendency of grate inlets to clog completely warrants consideration of a combination inlet, or curb-opening inlet in a sag where ponding can occur, and flanking inlets on the low gradient approaches.

3.5 Curb Inlet Design

3.5.1 Curb Inlets On Grade

Following is a discussion of the procedures for the design of curb inlets on grade. Curb opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb openings are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope, is determined using Figure 3-7. The efficiency of curb-opening inlets shorter than the length required for total interception, is determined using Figure 3-8.

The length of inlet required for total interception by depressed curb-opening inlets or curb-openings in depressed gutter sections can be found by the use of an equivalent cross slope, S_e, in the following equation.

$S_e = S_x + S'_w E_o$ (3.12)

Where: E_o = ratio of flow in the depressed section to total gutter flow

S'_w = cross slope of gutter measured from the cross slope of the pavement, S_x

$S'_w = (a/ 12W)$

Where: a = gutter depression, in

W = width of depressed gutter, ft

Design Steps
Steps for using Figures 3-7 and 3-8 in the design of curb inlets on grade are given below.
(1) Determine the following input parameters:
Cross slope = Sx (ft/ ft) Longitudinal slope = S (ft/ ft)
Gutter flow rate = Q (cfs) Manning's n = n Spread of water on pavement = T (ft) from Figure 3-1
(2) Enter Figure 3-7 using the two vertical lines on the left side labeled n and S. Locate the value for Manning's n and longitudinal slope and draw a line connecting these points and extend this line to the first turning line.
(3) Locate the value for the cross slope (or equivalent cross slope) and draw a line from the point on the first turning line through the cross slope value and extend this line to the second turning line.
(4) Using the far right vertical line labeled Q locate the gutter flow rate. Draw a line from this value to the point on the second turning line. Read the length required from the vertical line labeled LT.
(5) If the curb-opening inlet is shorter than the value obtained in step 4, Figure 3-8 can be used to calculate the efficiency. Enter the x-axis with the L/ LT ratio and draw a vertical line upward to the E curve. From the point of intersection, draw a line horizontally to the intersection with the y-axis and read the efficiency value.
Example
Given: Sx = 0.03 ft/ ft
S = 0.035 ft/ ft
S'w = 0. 083 (a = 2 in, W = 2 ft)
n = 0. 016
Q = 5 cfs
Find: (1) Qi for a 10-ft curb-opening inlet
(2) Qi for a depressed 10-ft curb-opening inlet with a = 2 in, W = 2 ft, T = 8 ft (Figure 3-1)
Solution: (1) From Figure 3-7, LT = 41 ft, L/ LT = 10/ 41 = 0. 24
From Figure 3-8, E = 0.39, Qi = EQ = 0. 39 x 5 = 2 cfs (2)
Qn = 5. 0 x 0. 016 = 0. 08 cfs
Sw/Sx = (0.03 + 0.083)/ 0.03 = 3.77
T/W = 3.5 (from Figure 3-3)
T = 3.5 x 2 = 7 ft
W/T = 2/7 = 0.29 ft
Eo = 0.72 (from Figure 3-2)
Therefore, Se = Sx + S'w Eo = 0.03 + 0.083(0.72) = 0.09
From Figure 3-7, LT = 23 ft, L/ LT = 10/ 23 = 0. 43
From Figure 3-8, E = 0.64, Qi = 0.64 x 5 = 3.2 cfs
The depressed curb-opening inlet will intercept 1.6 times the flow intercepted by the undepressed curb opening and over 60 percent of the total flow.

3.5.2 Curb Inlets In Sump

For the design of a curb-opening inlet in a sump location, the inlet operates as a weir to depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.
The capacity of curb-opening inlets in a sump location can be determined from Figure 3-9, which accounts for the operation of the inlet as a weir and as an orifice at depths greater than 1.4h. This figure is applicable to depressed curb-opening inlets and the depth at the inlet includes any gutter depression. The height (h) in the figure assumes a vertical

orifice opening (see sketch on Figure 3-9). The weir portion of Figure 3-9 is valid for a depressed curb-opening inlet when $d < (h + a/12)$.

The capacity of curb-opening inlets in a sump location with a vertical orifice opening but without any depression can be determined from Figure 3-10. The capacity of curb opening inlets in a sump location with other than vertical orifice openings can be determined by using Figure 3-11.

Design Steps

Steps for using Figures 3-9, 3-10, and 3-11 in the design of curb-opening inlets in sump locations are given below.

(1) Determine the following input parameters:

Cross slope = Sx (ft/ ft)

Spread of water on pavement = T (ft) from Figure 3-1

Gutter flow rate = Q (cfs) or dimensions of curb-opening inlet [L (ft) and H (in)]

Dimensions of depression if any [a (in) and W (ft)]

(2) To determine discharge given the other input parameters, select the appropriate Figure (3-9, 3-10, or 3-11 depending whether the inlet is in a depression and if the orifice opening is vertical).

(3) To determine the discharge (Q), given the water depth (d) locate the water depth value on the y-axis and draw a horizontal line to the appropriate perimeter (p), height (h), length (L), or width x length (hL) line. At this intersection draw a vertical line down to the x-axis and read the discharge value.

(4) To determine the water depth given the discharge, use the procedure described in step 3 except you enter the figure at the value for the discharge on the x-axis.

Example

Given: Curb-opening inlet in a sump location

$L = 5$ ft

$h = 5$ in

(1) Undepressed curb opening $Sx = 0.05$ ft/ ft

$T = 8$ ft

(2) Depressed curb opening

$Sx = 0.05$ ft/ ft

$a = 2$ in

$W = 2$ ft

$T = 8$ ft

Find: Discharge Qi

Solution: (1) $d = TSx = 8 \times 0.05 = 0.4$ ft

$d < h$

From Figure 3-10, $Qi = 3.8$ cfs

(2) $d = 0.4$ ft

$h + a/12 = (5 + 2/12)/12 = 0.43$ ft

since $d < 0.43$ the weir portion of Figure 3-9 is applicable (lower portion of the Figure).

$P = L + 1.8W = 5 + 3.6 = 8.6$ ft

From Figure 3-9, $Qi = 5$ cfs

At $d = 0.4$ ft, the depressed curb-opening inlet has about 30 percent more capacity than an inlet without depression.

3.6 Combination Inlets

3.6.1 On Grade

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located adjacent to the grate is approximately equal to the capacity of the grate inlet alone. Thus capacity is computed by neglecting the curb-opening inlet and the design procedures should be followed based on the use of Figures 3-4, 3-5 and 3-6.

3.6.2 In Sump

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point, or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity. Assuming complete clogging of the grate, Figures 3-9, 3-10, and 3-11 for curb-opening inlets should be used for design. Note, all drains will be appropriately marked or stenciled for illicit discharge.

3.7 Storm Drains

3.7.1 Introduction

After the tentative locations of inlets, drain pipes, and outfalls with tail-waters have been determined and the inlets sized, the next logical step is the computation of the rate of discharge to be carried by each drain pipe and the determination of the size and gradient of pipe required to care for this discharge. This is done by proceeding in steps from upstream of a line to downstream to the point at which the line connects with other lines or the outfall, whichever is applicable. The discharge for a run is calculated, the drainpipe serving that discharge is sized, and the process is repeated for the next run downstream. It should be recognized that the rate of discharge to be carried by any particular section of drain pipe is not necessarily the sum of the inlet design discharge rates of all inlets above that section of pipe, but as a general rule is somewhat less than this total. It is useful to understand that the time of concentration is most influential and as the time of concentration grows larger, the proper rainfall intensity to be used in the design grows smaller.

For ordinary conditions, drainpipes should be sized on the assumption that they will flow full or practically full under the design discharge but will not be placed under pressure head. The Manning Formula is recommended for capacity calculations.

3.7.2 Design Criteria

The standard recommended maximum and minimum slopes for storm drains should conform to the following criteria:

1. The maximum hydraulic gradient should not produce a velocity that exceeds 15 feet per second.
2. The minimum desirable physical slope should be 0.5 percent or the slope, which will produce a velocity of 3.0 feet per second when the storm sewer is flowing full, whichever is greater.

If the potential water surface elevation exceeds one foot below ground elevation for the design flow, the top of the pipe, or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the hydraulic grade line.

3.7.3 Capacity

Formulas for Gravity and Pressure Flow

The most widely used formula for determining the hydraulic capacity of storm drain pipes for gravity and pressure flows is the Manning Formula, expressed by the following equation:

$$V = [1.486\, R^{2/3} S^{1/2}]/n \quad (3.13)$$

Where: V = mean velocity of flow, ft/s

R = the hydraulic radius, ft -defined as the area of flow divided by the wetted flow surface or wetted perimeter (A/ WP)

S = the slope of hydraulic grade line, ft/ft

n = Manning's roughness coefficient

In terms of discharge, the above formula becomes:

$$Q = [1.486\, AR^{2/3} S^{1/2}]/n \quad (3.14)$$

Where: Q = rate of flow, cfs

A = cross sectional area of flow, ft^2

For pipes flowing full, the above equations become:

$$V = [0.590\, D^{2/3} S^{1/2}]/n \quad (3.15)$$

$$Q = [0.463\, D^{8/3} S^{1/2}]/n \quad (3.16)$$

Where: D = diameter of pipe, ft

The Manning's equation can be written to determine friction losses for storm drain pipes as:

$$H_f = [2.87\, n^2 V^2 L]/[S^{4/3}] \quad (3.17)$$

$$H_f = [29\, n^2 V^2 L]/[(R^{4/3})(2g)] \quad (3.18)$$

Where: H_f = total head loss due to friction, ft

n = Manning's roughness coefficient

D = diameter of pipe, ft

L = length of pipe, ft

V = mean velocity, ft/s

R = hydraulic radius, ft

g = acceleration of gravity = 32.2 ft/sec^2

3.7.4 Nomographs And Table

The nomograph solution of Manning's formula for full flow in circular storm drainpipes is shown on Figures 3-12, 3-13, and 3-14. Figure 3-15 has been provided to solve the Manning's equation for part full flow in storm drains.

3.7.5 Hydraulic Grade Lines

All head losses in a storm sewer system are considered in computing the hydraulic grade

line to determine the water surface elevations, under design conditions in the various inlets, catch basins, manholes, junction boxes, etc.
Hydraulic control is a set water surface elevation from which the hydraulic calculations are begun. All hydraulic controls along the alignment are established. If the control is at a main line upstream inlet (inlet control), the hydraulic grade line is the water surface elevation minus the entrance loss minus the difference in velocity head. If the control is at the outlet, the water surface is the outlet pipe hydraulic grade line.
Design Procedure -Outlet Control. The head losses are calculated beginning from the control point to the first junction and the procedure is repeated for the next junction. The computation for an outlet control may be tabulated on Figure 3-16 using the following procedure:
1. Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. Hydraulic grade line computations begin at the outfall and are worked
2. Enter in Col. 2 the outlet water surface elevation if the outlet will be submerged during the design storm or 0.8 diameter plus invert elevation of the outflow pipe whichever is greater.
3. Enter in Col. 3 the diameter (D_o) of the outflow pipe.
4. Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.
5. Enter in Col. 5 the length (L_o) of the outflow pipe.
6. Enter in Col. 6 the friction slope (S_f) in ft/ ft of the outflow pipe. This can be determined by using the following formula:
$S_f = (Q^2)/K$ (3. 19)
Where: S_f = friction slope
$K = [1.486\ AR^{2/3}]/n$
7. Multiply the friction slope (S_f) in Col. 6 by the length (L_o) in Col. 5 and enter the friction loss (H_f) in Col. 7. On curved alignments, calculate curve losses by using the formula $H_c = 0.002\tilde{()}(V_o^2/2g)$, where = angle of curvature in degrees and add to the friction loss.
8. Enter in Col. 8 the velocity of the flow (V_o) of the outflow pipe.
9. Enter in Col. 9 the contraction loss (H_o) by using the formula:
$H_o = [0.25\ V_o^2)]/2g$
Where g = 32. 2-ft/ s2
10. Enter in Col. 10 the design discharge (Q_i) for each pipe flowing into the junction. Neglect lateral pipes with inflows of less than ten percent of the mainline outflow. Inflow must be adjusted to the mainline outflow duration time before a comparison is made.
11. Enter in Col. 11 the velocity of flow (V_i) for each pipe flowing into the junction (for exception see Step 10).
12. Enter in Col. 12 the product of $Q_i \times V_i$ for each inflowing pipe. When several pipes inflow into a junction, the line producing the greatest $Q_i \times V_i$ product is the one that should be used for expansion loss calculations.
13. Enter in Col. 13 the controlling expansion loss (H_i) using the formula: $H_i = [0.35\ (V_1^2)]/2g$.
14. Enter in Col. 14 the angle of skew of each inflowing pipe to the outflow pipe (for exception, see Step 10).
15. Enter in Col. 15 the greatest bend loss (H) calculated by using the formula $H = [KV_i^2)]/2g$ where K = the bend loss coefficient corresponding to the various angles of skew of the inflowing pipes.
16. Enter in Col. 16 the total head loss (H_t) by summing the values in Col. 9 (H_o), Col. 13 (H_i), and Col. 15 ($\tilde{H}$).
17. If the junction incorporates adjusted surface inflow of ten percent or more of the

mainline outflow, i.e., drop inlet, increase H_t by 30 percent and enter the adjusted H_t in Col. 17.

18. If the junction incorporates full diameter inlet shaping, such as standard manholes, reduce the value of H_t by 50 percent and enter the adjusted value in Col. 18.

19. Enter in Col. 19 the FINAL H, the sum of H_f and H_t, where H_t is the final adjusted value of the H_t.

20. Enter in Col. 20 the sum of the elevation in Col. 2 and the Final H in Col. 19. This elevation is the potential water surface elevation for the junction under design conditions.

21. Enter in Col. 21 the rim elevation or the gutter flow line, whichever is lowest, of the junction under consideration in Col. 20. If the potential water surface elevation exceeds one foot below ground elevation for the design flow, the top of the pipe or the gutter flow line, whichever is lowest, adjustments are needed in the system to reduce the elevation of the Hydraulic Grade Line (H. G. L.).

22. Repeat the procedure starting with Step 1 for the next junction upstream.

23. At last upstream entrance, add $V_1^2/2g$ to get upstream water surface elevation.

3.7.6 Minimum Grade

All storm drains should be designed such that velocities of flow will not be less than 3.0 feet per second at design flow or lower, with a minimum slope of 0.5 percent. For very flat flow lines the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. Upper reaches of a storm drain system should have flatter slopes than slopes of lower reaches. Progressively increasing slopes keep solids moving toward the outlet and deters settling of particles due to steadily increasing flow streams.

The minimum slopes are calculated by the modified Manning formula:

$$S = [(nV)^2]/[2.208R^{4/3}] \quad (3.20)$$

Where: S = the slope of the hydraulic grade line, ft/ ft
n = Manning's roughness coefficient
V = mean velocity of flow, ft/ s
R = hydraulic radius, ft (area divided by wetted perimeter)

3.7.7 Storm Drain Storage

If downstream drainage facilities are undersized for the design flow, an above-or belowground detention structure may be needed to reduce the possibility of flooding. Using larger than needed storm drain pipes sizes and restrictors to control the release rates at manholes and/ or junction boxes in the storm drain system can provide the required storage volume. The same design criteria for sizing the detention basin is used to determine the storage volume required in the system.

3.7.8 Design Procedures

The design of storm drain systems is generally divided into the following operations:

1. The first step is the determination of inlet location and spacing as outlined earlier in this chapter.
2. The second step is the preparation of a plan layout of the storm sewer drainage system establishing the following design data:
a. Location of storm drains.
b. Direction of flow.
c. Location of manholes.
d. Location of existing facilities such as water, gas, or underground cables.
3. The design of the storm drain system is then accomplished by determining

drainage areas, computing runoff by rational method, and computing the hydraulic capacity by Manning's equation.

4. The storm drain design computation sheet (Figure 3-17) can be used to summarize the hydrologic, hydraulic and design computations.

5. Examine all assumptions to determine if any adjustments are needed to the final design.

3.8 References

U. S. Department of Transportation, Federal Highway Administration, 1984. Drainage of Highway Pavements. Hydraulic Engineering Circular No. 12.

Atlanta Regional Commission, Georgia Stormwater Design Manual. 2000.

4 Culvert Design

4.1 Introduction

Primary considerations for the final selection of any drainage structure are that its design be based upon appropriate hydraulic principles, economy, and minimized effects on adjacent property by the resultant headwater depth and outlet velocity. The allowable headwater elevation is that elevation above which damage may be caused to adjacent property and/ or the highway. It is this allowable headwater depth that is the primary basis for sizing a culvert.

Performance curves should be developed for all culverts for evaluating the hydraulic capacity of a culvert for various headwaters. These will display the consequence of high flow rates at the site and any possible hazards. Sometimes a small increase in flow rate can affect a culvert design. If only the design peak discharge is used in the design, the engineer cannot assess what effect increases in the estimated design discharge will have on the culvert design.

4.2 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in many culvert design publications.

4.3 Engineering Design Criteria

The design of a culvert should take into account many different engineering and technical aspects at the culvert site and adjacent areas. The following design criteria should be considered for all culvert designs as applicable.

4.3.1 Frequency Flood

The design storm for a culvert for all roads is the 25-year storm using future development land use conditions, assuming no detention. The design of lateral systems shall be based on a 25-year storm event, using future development land use conditions assuming no detention. The 100-year frequency storm shall be routed through all culverts to be sure building structures (i.e., houses, commercial buildings) are not flooded or increased damage does not occur to the highway or adjacent property for this design event.

Table 4-1 Symbols And Definitions

Symbol Definition Units

A
B
Cd
D
d
dc
du
g
Hf
ho
HW
Ke
L
N
Q
S
TW
V
Vc

Area of cross section of flow
Barrel width
Overtopping discharge Coefficient
Culvert diameter of barrel depth
Depth of flow
Critical depth of flow
Uniform depth of flow
Acceleration of gravity
Depth of pool or head, above the face section of invert
Height of hydraulic grade line above outlet invert
Headwater depth above invert of culvert (depth from inlet invert to upstream total energy grade line)
Inlet loss coefficient
Length of culvert
Number of barrels
Rate of discharge
Slope of culvert
Tailwater depth above invert of culvert
Mean velocity of flow
Critical velocity
sq. ft
ft
-
in. or ft
ft
ft
ft
ft/s
ft
ft
ft
-
ft
-
cfs
ft/f
ft
ft/s
ft/s

4.3.2 Velocity Limitations

Both minimum and maximum velocities should be considered when designing a culvert. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. The maximum allowable velocity for corrugated metal pipe is 15 feet per second. There is no specified maximum allowable velocity for reinforced concrete pipe, but outlet protection shall be provided where discharge velocities will cause erosion problems. To ensure self-cleaning during partial depth flow, a minimum velocity of 3.0 feet per second, for the 2-year flow, when the culvert is flowing partially full is required.

4.3.3 Buoyancy Protection

Headwalls, endwalls, slope paving or other means of anchoring to provide buoyancy protection should be considered for all flexible culverts.

4.3.4 Length And Slope

The culvert length and slope should be chosen to approximate existing topography, and to the degree practicable: the culvert invert should be aligned with the channel bottom and the skew angle of the stream, and the culvert entrance should match the geometry of the roadway embankment. The maximum slope using concrete pipe is 10% and for CMP is 14% before pipe-restraining methods must be taken. Maximum drop in a drainage structure is 10 feet.

4.3.5 Debris Control

In designing debris control structures it is recommended that the Hydraulic Engineering Circular No. 9 entitled "Debris -Control Structures" be consulted.

4.3.6 Headwater Limitations

The allowable headwater elevation is determined from an evaluation of land use upstream of the culvert and the proposed or existing roadway elevation. Headwater is the depth of water above the culvert invert at the entrance end of the culvert.

The following criteria related to headwater should be considered:

- The allowable headwater for design frequency conditions should allow for the following upstream controls.

o 18-inch freeboard.

- Minimize upstream property damage.
- Elevations established to delineate flood plain zoning.
- Low point in the road grade that is not at the culvert location.
- Ditch elevation of the terrain that will permit flow to divert around culvert.

o Following HW/ D criteria –

(1) For drainage facilities with cross-section area equal to or less than 30 sq. ft -HW/ D = to or < 1. 5.

(2) For drainage facilities with cross-section area greater than 30 sq. ft -HW/ D = to or < 1. 2.

o The headwater should be checked for the 100-year flood to ensure compliance with flood plain management criteria and for most facilities the culvert should be sized to maintain flood-free conditions on major thoroughfares with 18 inches freeboard at the low-point of the road.

o The maximum acceptable outlet velocity should be identified (see Section 4. 3 in the Open Channel Conveyance Design Chapter). Either the headwater should be set to produce acceptable velocities or stabilization or energy dissipation should be provided where these velocities are exceeded.

o Other site-specific design considerations should be addressed as required.

o In general the constraint which gives the lowest allowable headwater elevation establishes the criteria for the hydraulic calculations.

4.3.7 Tailwater Considerations

The hydraulic conditions downstream of the culvert site must be evaluated to determine a tailwater depth for a range of discharge. At times there may be a need for calculating backwater curves to establish the tailwater conditions. The following conditions must be considered:

- If the culvert outlet is operating with a free outfall, the critical depth and equivalent hydraulic grade line should be determined.
- For culverts, which discharge to an open channel, the stage-discharge curve for the channel must be determined. See Open Channel Conveyance Design Chapter.
- If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish the design tailwater depth for the upstream culvert.

If the culvert discharges to a lake, pond, or other major water body, the expected high water elevation of the particular water body may establish the culvert tailwater.

4.3.8 Storage

If storage is being assumed upstream of the culvert, consideration should be given to:

- the total area of flooding,
- the average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, an
- ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

4.3.9 Culvert Inlets

Hydraulic efficiency and cost can be significantly affected by inlet conditions. The inlet coefficient Ke , is a measure of the hydraulic efficiency of the inlet, with lower values indicating greater efficiency. Recommended inlet coefficients are given in Table 4-2.

4.3.10 Inlets With Headwalls

Headwalls may be used for a variety of reasons including increasing the efficiency of the inlet, providing embankment stability, providing embankment protection against erosion, providing protection from buoyancy, and shorten the length of the required structure. Headwalls are required for all metal culverts and where buoyancy protection is necessary. If high headwater depths are to be encountered, or the approach velocity in the channel will cause scour, a short channel apron should be provided at the toe of the headwall. This apron should extend at least one pipe diameter upstream from the entrance, and the top of the apron should not protrude above the normal streambed elevation.

4.3.11 Wingwalls And Aprons

Wingwalls are used where the side slopes of the channel adjacent to the entrance are unstable or where the culvert is skewed to the normal channel flow.

4.3.12 Improved Inlets

Where inlet conditions control the amount of flow that can pass through the culvert, improved inlets can greatly increase the hydraulic performance at the culvert.

4.3.13 Material Selection

Reinforced concrete pipe (RCP) is recommended for use (1) under a roadway, (2) when pipe slopes are less than 1%, or (3) for all flowing streams. High Density Polyethylene pipe may also be used as specified in the municipal regulations. Table 4-3 gives recommended Manning's n values for different materials.

4.3.14 Culvert Skews

Culvert skews shall not exceed 45 degrees as measured from a line perpendicular to the roadway centerline without approval.

4.3.15 Culvert Sizes

The minimum allowable pipe diameter shall be 18 inches.

4.3.16 Weep Holes

Weep holes are sometimes used to relieve uplift pressure. Filter materials should be used in conjunction with weep holes in order to intercept the flow and prevent the formation of piping channels. The filter material should be designed as underdrain filter so that it will not become clogged and so that piping cannot occur through the pervious material and the weep holes.

4.3.17 Outlet Protection

See Energy Dissipation Designs Chapter for information on the design of outlet protection. Outlet protection should be provided for the 25-year storm.

4.3.18 Erosion And Sediment Control

Erosion and sediment control shall be in accordance with the latest approved Manual For Erosion and Sediment Control in Georgia for design standards and details related to erosion and sediment control.

4.3.19 Environmental Considerations

Where compatible with good hydraulic engineering, a site should be selected that will permit the culvert to be constructed to cause the least impact on the stream or wetlands. This selection must consider the entire site, including any necessary lead channels.

Table 4-2 Inlet Coefficients

Type of Structure and Design of Entrance Coefficient Ke

Pipe, Concrete
Projecting from fill, socket end (groove-end)
Projecting from fill, square cut end
Headwall or headwall and wingwalls
0.2
0.5
Socket end of pipe (groove end)
Square-edge
Rounded [radius = 1/12(D)]
0.2
0.5
0.2
Mitered to conform to fill slope
*End section conforming to fill slope
Beveled edges, 33.7o or 45o bevels
Side-or slope-tapered inlet
0.7
0.5
0.2
0.2
Pipe, or Pipe-Arch, Corrugated Metal1
Projecting from fill (no headwall)
Headwall or headwall and wingwalls square-edge
Mitered to fill slope, paved or unpaved slope
*End-Section conforming to fill slope
Beveled edges, 33.7o or 45o bevels
Side-or slope-tapered inlet
0.9
0.5
0.7
0.5
0.2
0.2
Box, Reinforced Concrete
Headwall parallel to embankment (no wingwalls)
Square-edged on 3 sides
Rounded on 3 edges to radius of [1/12(D)]
or beveled edges on 3 sides
Wingwalls at 30o or 75o to barrel
Square-edged at crown
Crown edge rounded to radius of [1/12(D)]
or beveled top edge
Wingwalls at 10o or 25o to barrel
Square-edged at crown
Wingwalls parallel (extension of sides)
Square-edged at crown

Side-or slope-tapered inlet
0.5
0.2
0.4
0.2
0.5
0.7
0.2
Table 4-3 Manning's n Values
Type of Conduit Wall & Joint Description Manning's n
Concrete Pipe Good joints, smooth walls
Good joints, rough walls
Poor joints, rough walls
Concrete Box Good joints, smooth finished walls
Poor joints, rough, unfinished walls
2 2/3 by ½ inch corrugations
6 by 1 inch corrugations
5 by 1 inch corrugations
3 by 1 inch corrugations
6 by 2 inch structural plate
9 by 2 ½ inch structural plate
Corrugated Metal Pipes
and Boxes Angular
Corrugations Pipes,
Helical Corrugations,
Full
Circular Flow Spiral
Rib Metal Pipe
High Density
Polyethylene Polyvinyl
Chloride 2 2/3 by ½ inch corrugated 24 inch plate
0.012
0.016
0.017
0.012
0.018
0.024
0.025
0.026
0.028
0.035
0.035
0.012
Note: For further information concerning Manning n values for selected conduits consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163 in conjunction with the weep holes in order to intercept the flow and prevent the formation of piping channels. The filter materials should be designed as an underdrain filter so that it will not become clogged and so that piping cannot occur through the pervious material and the weep hole.
4.4 Design Procedures
4.4.1 Inlet And Outlet Control

Inlet Control -If the culvert is operating on a steep slope it is likely that the entrance geometry will control the headwater and the culvert will be on inlet control.
Outlet Control -If the culvert is operating on a mild slope, the outlet characteristics will probably control the flow and the culvert will be on outlet control.
Proper culvert design and analysis requires checking for both inlet and outlet control to determine which will govern particular culvert designs. For more information on inlet and outlet control see the Federal Highway Administration publication entitled - Hydraulic Design Of Highway Culverts, HDS-5, 1985, and AASHTO Model Drainage Manual, 1998.

4.4.2 Procedures

There are two procedures for designing culverts: (1) the manual use of inlet and outlet control nomographs and (2) the use of a personal computer system such as HY8 -Culvert Analysis Microcomputer Program.
It is recommended that the HY8 computer model be used for culvert design. The personal computer system HYDRAIN, which includes HY8, uses the theoretical basis for the nomographs to size a culvert. Other computer programs can be used if approved by the municipality. In addition, this system can evaluate improved inlets, route hydrographs, consider road overtopping, and evaluate outlet streambed scour. By using water surface profiles, this procedure is more accurate in predicting backwater effects and outlet scour.
The following will outline the design procedures for use of the nomograph.

4.4.3 Nomographs

The use of nomographs require a trial and error solution. The solution is quite easy and provides reliable designs for many applications. It should be remembered that velocity, hydrograph routing, roadway overtopping, and outlet scour require additional, separate computations beyond what can be obtained from the nomographs.
Figures 4-1 and 4-2 show examples of an inlet control and outlet control nomograph that can be used to design concrete pipe culverts. For culvert designs not covered by these nomographs, refer to the complete set of nomographs given in the Manual for Erosion and Sediment Control in Georgia, Appendix A-1.

4.4.4 Steps In Design Procedure

The design procedure requires the use of inlet and outlet nomographs.
Step Action
(1) List design data:
Q = discharge (cfs) L = culvert length (ft)
S = culvert slope (ft/ ft) TW= tailwater depth (ft)
V = velocity for trial diameter (ft/ s) Ke = inlet loss coefficient
HW= allowable headwater depth for the design storm (ft)
(2) Determine trial culvert size by assuming a trial velocity 3 to 5 ft/ s and computing the culvert area, A = Q/ V. Determine the culvert diameter (inches).
(3) Find the actual HW for the trial size culvert for both inlet and outlet control.

- For inlet control, enter inlet control nomograph with D and Q and find HW/ D for the proper entrance type.
- Compute HW and, if too large or too small, try another culvert size before computing HW for outlet control.
- For outlet control enter the outlet control nomograph with the culvert length, entrance loss coefficient, and trial culvert diameter.
- To compute HW, connect the length scale for the type of entrance condition and culvert diameter scale with a straight line, pivot on the turning line, and draw a straight line from the design discharge through the

turning point to the head loss scale H. Compute the headwater elevation HW from the equation:

HW = H + ho -LS (4. 1)

Where: ho = ½ (critical depth + D), or tailwater depth, whichever is greater.

(4) Compare the computed headwaters and use the higher HW nomograph to determine if the culvert is under inlet or outlet control.

If outlet control governs and the HW is unacceptable, select a larger trial size and find another HW with the outlet control nomographs. Since the smaller size of culvert had been selected for allowable HW by the inlet control nomographs, the inlet control for the larger pipe need not be checked.

(5) Calculate exit velocity and expected streambed scour to determine if an energy dissipator is needed.

4.4.5 Performance Curves -Roadway Overtopping

A performance curve for any culvert can be obtain from the nomographs by repeating the steps outlined above for a range of discharges that are of interest for that particular culvert design. A graph is then plotted of headwater vs. discharge with sufficient points so that a curve can be drawn through the range of interest. These curves are applicable through a range of headwater, velocities, and scour depths versus discharges for a length and type of culvert. Usually charts with length intervals of 25 to 50 feet are satisfactory for design purposes. Such computations are made much easier by the computer program discussed in the next section of this manual.

To complete the culvert design, roadway overtopping should be analyzed. A performance curve showing the culvert flow as well as the flow across the roadway is a useful analysis tool. Rather than using a trial and error procedure to determine the flow division between the overtopping flow and the culvert flow, an overall performance curve can be developed.

The overall performance curve can be determined as follows:

Step Action

(1) Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. The flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters should be calculated.

(2) Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.

(3) When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the equivalent upstream water surface depth above the roadway (crest of weir) for each selected flow rate. Use these water surface depths and equation 4.2 to calculate flow rates across the roadway.

Q = Cd L(HW)1.5 (4.2)

Where: Q = overtopping flow rate (ft^3/s)

Cd = overtopping discharge coefficient

L = length of roadway (ft)

HW = upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown (ft)

Note: See Figure 4-3 for guidance in determining a value for Cd . For more information on calculating overtopping flow rates see pages 39 -42 in HDS No. 5.

(4) Add the culvert flow and the roadway overtopping flow at the

corresponding headwater elevations to obtain the overall culvert performance curve.

4.4.6 Storage Routing

A significant storage capacity behind a highway embankment attenuates a flood hydrograph. Because of the reduction of the peak discharge associated with this attenuation, the required capacity of the culvert, and its size, may be reduced considerably. If significant storage is anticipated behind a culvert, the design should be checked by routing the design hydrographs through the culvert to determine the discharge and stage behind the culvert. Routing procedures are outlined in Hydraulic Design of Highway Culverts, Section V -Storage Routing, HDS No. 5, Federal Highway Administration.

The storage should be taken into consideration only if the storage area will remain available for the life of the culvert as a result of purchase of ownership or right-of-way or an easement has been acquired.

4.5 Culvert Design Example

4.5.1 Introduction

The following example problem illustrates the procedures to be used in designing culverts using the nomographs.

4.5.2 Example

Size a culvert given the following example data, which were determined by physical limitations at the culvert site and hydraulic procedures described elsewhere in this handbook.

4.5.3 Example Data

Input Data

Discharge for 2-yr flood = 35 cfs

Discharge for 25-yr flood = 70 cfs

Allowable HW for 25-yr discharge = 7.0 ft

Length of culvert = 100 ft

Natural channel invert elevations -inlet = 15.50 ft, outlet = 15. 35 ft

Culvert slope = 0. 0015 ft/ ft

Tailwater depth for 25-yr discharge = 4.0 ft

Tailwater depth is the normal depth in downstream channel

Entrance type = Groove end with headwall

4.5.4 Computations

Steps Computation

1. Assume a culvert velocity of 5-ft/ s. Required flow area = 70 cfs/ 5 ft/s = 14 sq. ft (for the 25-yr recurrence flood).
2. The corresponding culvert diameter is about 48 in. This can be calculated by using the formula for area of a circle: Area = (3.14D2)/ 4 or D = (Area times 4/ 3.14)0.5. Therefore: D = ((14 sq ft x 4)/ 3.14)0.5 x 12 in./ft) = 50.7 in.
3. A grooved end culvert with a headwall is selected for the design. Using the inlet control nomograph (Figure 4-1), with a pipe diameter of 48 in. and a discharge of 70 cfs; read a HW/D value of 0.93.
4. The depth of headwater (HW) is (0.93) x (4) = 3.72 ft which is less than the allowable headwater of 4.5 ft.
5. The culvert is checked for outlet control by using Figure 4-2. With an entrance loss coefficient Ke of 0.20, a culvert length of 100 ft, and a pipe diameter of 48 in., an H value of 0.77 ft are determined. The headwater for outlet control is computed by the equation: HW = H + ho LS

For the tailwater depth lower than the top of culvert,

ho = TW or ½ (critical depth in culvert + D) whichever is greater.
ho = 3.0 ft or ho = ½ (2.55 + 4.0) = 3.28 ft
The headwater depth for outlet control is:
HW = H + ho -LS = 0.77 + 3.28 -(100) x (0.0015) = 3.90 ft
6. Since HW for outlet control (3.90 ft) is greater than the HW for inlet control (3.72 ft), outlet control governs the culvert design. Thus, the maximum headwater expected for a 25-yr recurrence flood is 3.90 ft, which is less than the allowable headwater of 4.5 ft.
7. Estimate outlet exit velocity. Since this culvert is on outlet control and discharges into an open channel downstream, the culvert will be flowing full at the flow depth in the channel. Using the design peak discharge of 70 cfs and the area of a 48 in. or 4.0 ft diameter culvert the exit velocity will be:
Q = VA Therefore:
V = 70 / (3.14(4.0)2)/ 4 = 5.6 ft/ s
8. Check for minimum velocity using the 2-year flow of 35 cfs.
Therefore: V = 35 / (3.14(4.0)2)/ 4 = 2.8 ft/ s > minimum of 2.5 -OK
9. The 100-year flow should be routed through the culvert to determine if any flooding problems will be associated with this flood.
Figure 4-4 on the next page provides a convenient form to organize culvert design calculations.

4.6 Design Procedures For Beveled-Edged Inlets

4.6.1 Introduction

Improved inlets include inlet geometry refinements beyond those normally used in conventional culvert design practice. Several degrees of improvements are possible, including bevel-edged, side-tapered, and slope-tapered inlets. Those designers interested in using side-and slope-tapered inlets should consult the detailed design criteria and example designs outlined in the U. S. Department of Transportation publication Hydraulic Engineering Circular No. 5 entitled, Hydraulic Design of Highway Culverts.

4.6.2 Design Figures

Four inlet control figures for culverts with beveled edges are included in the hydrograph tables.

Chart Use for -
3
10
11
12
Circular pipe culverts with beveled rings
90o headwalls (same for90o wingwalls)
skewed headwalls
wingwalls with flare angles of 18 to 45

The following symbols are used in these figures:
B -Width of culvert barrel or diameter of pipe culvert
D -Height of box culvert or diameter of pipe culvert
Hf -Depth of pool or head, above the face section of invert
N -Number of barrels
Q -Design discharge

4.6.3 Design Procedure

The figures for bevel-edged inlets are used for design in the same manner as the conventional inlet design nomographs discussed earlier. Note that Charts 10, 11, and 12 in Appendix A apply only to bevels having either a 33o angle (1.5:1) or a 45o angle (1:1).

For box culverts the dimensions of the bevels to be used are based on the culverts dimensions. The top bevel dimension is determined by multiplying the height of the culvert by a factor. The side bevel dimensions are determined by multiplying the width of the culvert by a factor. For a 1:1 bevel, the factor is ½ inch/ft. For a 1.5:1 bevel the factor is 1 inch/ ft. For example the minimum bevel dimensions for an 8 ft x 6 ft box culvert with 1:1 bevels would be:

Top Bevel = d = 6 ft x ½ inch/ ft = 3 inches

Side Bevel = b = 8 ft x ½ inch/ ft = 4 inches

For a 1.5:1 bevel computations would result in d = 6 and b = 8 inches.

4.6.4 Design Figure Limits

The improved inlet design figures are based on research results from culvert models with barrel width, B, to depth, D, ratios of from 0.5:1 to 2:1. For box culverts with more than one barrel, the figures are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size.

For example, in a double 8 ft by 8 ft box culvert:

Top Bevel -is proportioned based on the height of 8 ft which results in a bevel of 4 in. for the 1:1 bevel and 8 in. for the 1.5:1 bevel.

Side Bevel -is proportioned based on the clear width of 16 ft which results in a bevel of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel.

4.6.5 Multibarrel Installations

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller.

The top bevel dimension should always be based on the culvert height.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as the edge condition of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half of the thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to help direct debris through the culvert.

Multibarrel pipe culverts should be designed as a series of single barrel installations since each pipe requires a separate bevel.

4.6.6 Skewed Inlets

It is recommended that Chart 11 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in under design due to a greatly reduced capacity. Skewed inlets (at an angle with the centerline of the stream) should be avoided whenever possible, and should not be used with side-or slope-tapered inlets. It is important to align culverts with streams in order to avoid erosion problems associated with changing the direction of the natural stream flow.

4.7 Flood Routing And Culvert Design

4.7.1 Introduction

Flood routing through a culvert is a practice that evaluates the effect of temporary upstream ponding caused by the culvert's backwater. By not considering flood routing it is possible that the findings from culvert analyses will be conservative. If the selected allowable headwater is considered acceptable without flood routing, then costly over design of both the culvert and outlet protection may result, depending on the amount of temporary storage involved. However, if storage is used in the design of culverts, consideration should be given to:

- the total area of flooding,
- the average time that bankfull stage is exceeded for the design flood up to 48 hours in rural areas or 6 hours in urban areas, and
- ensuring that the storage area will remain available for the life of the culvert through the purchase of right-of-way or easement.

Ignoring temporary storage effects on reducing the selected design flood magnitude by assuming that this provides a factor of safety is not recommended. This practice results in inconsistent factors of safety at culvert sites as it is dependent on the amount of temporary storage at each site. Further, with little or no temporary storage at a site the factor of safety would be unity thereby precluding a factor of safety. If a factor of safety is desired, it is essential that flood routing practices be used to insure consistent and defensible factors of safety are used at all culvert sites.

4.7.2 Design Procedure

The design procedure for flood routing through a culvert is the same as for reservoir routing. The site data and roadway geometry are obtained and the hydrology analysis completed to include estimating a hydrograph. Once this essential information is available, the culvert can be designed. Flood routing through a culvert can be time consuming. It is recommended that the HY8 computer program be used as it contains software that very quickly routes floods through a culvert to evaluate an existing culvert (review), or to select a culvert size that satisfies given criteria (design). However, the engineer should be familiar with the culvert flood routing design process.

A Multiple trial and error procedure is required for culvert flood routing. In general:

- a trial culvert(s) is selected,
- a trial discharge for a particular hydrograph time increment (selected time increment to estimate discharge from the design hydrograph) is selected,
- flood routing computations are made with successive trial discharges until the flood routing equation is satisfied,
- the hydraulic findings are compared to the selected site criteria, and
- if the selected site criteria are satisfied then a trial discharge for the next time increment is selected and this procedure is repeated; if not, a new trial culvert is selected and the entire procedure is repeated.

4.8 HY8 Culvert Analysis Microcomputer Program

Although the charts and nomographs given in this chapter can be used for culvert design, many designs require roadway overtopping analysis and other analysis, which makes the computation tedious. For these reasons most designers used computer programs for culvert design and analysis. The HY8 culvert analysis microcomputer program is one very popular culvert design and analysis program that will perform the calculations for the following:

1. culvert analysis (including independent multiple barrel sizing)
2. hydrograph generation
3. hydrograph routing
4. roadway overtopping
5. outlet scour estimates

4.9 References

American Association of State Highway and Transportation Officials. 1982. Highway Drainage Guidelines.

American Association of State Highway and Transportation Officials. 1998. Model Drainage Manual.

Debo, Thomas N. and Andrew J. Reese. Municipal Stormwater Management. Lewis Publishers. 1995.

Federal Highway Administration. 1978. Hydraulics of Bridge Waterways. Hydraulic

Design Series No. 1.
Federal Highway Administration. 1985. Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5.
Federal Highway Administration. 1971. Debris-Control Structures. Hydraulic Engineering Circular No. 9.
Federal Highway Administration. 1987. HY8 Culvert Analysis Microcomputer Program Applications Guide. Hydraulic Microcomputer Program HY8.
Federal Highway Administration. 1996. Urban Drainage Design Manual. Hydraulic Engineering Circular No. 22.
HYDRAIN Culvert Computer Program (HY8). Available from McTrans Software, University of Florida, 512 Weil Hall, Gainesville, Florida 32611.
U. S. Department of Interior. 1983. Design of Small Canal Structures.

5 Open Channel Hydraulics

5.1 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in open channel publications.

Table 5-1 Symbols and Definitions

Symbol Definition Units

□
A
b
Cg
D or d
d
deltad
dx
E
Fr
g
hloss
K
Ke
KT
L
Lp
n
P
Q
R
Rc
S
SWs
T
V or v
w
yc
yn
z
Energy coefficient
Cross-sectional area
Bottom width
Specific weight correction factor
Depth of flow
Stone diameter
Superelevation of the waters surface profile
Diameter of stone for which x percent, by weight,
of the gradation is finer
Specific Energy
Froude Number
Acceleration of gravity
Head loss

Channel conveyance
Eddy head loss coefficient
Trapezoidal open channel conveyance factor
Length of channel
Length of downstream protection
Manning's roughness coefficient
Wetted perimeter
Discharge rate
Hydraulic radius of flow
Mean radius of the bend
Slope
Specific Weight of stone
Top width of waters surface
Velocity of flow
Stone weight
Critical depth
Normal depth
Critical flow section factor
ft2
ft
ft
ft
ft
ft
ft
32.2 ft/s2
ft
ft
ft
ft
ft
cfs
ft
ft
ft/ft
lbs/ft3
ft
ft/s
lbs
ft
ft

5.2 Design Criteria

5.2.1 General Criteria

In general, the following criteria should be used for open channel design:

1. Channel side slopes shall be stable throughout the entire length and side slope shall depend on the channel material. A maximum of 2: 1 should be used for channel side slopes, unless otherwise justified by calculations. Roadside ditches should have a maximum slope of 3:1.
2. Trapezoidal or parabolic cross sections are preferred over triangular shapes.
3. For vegetative channels, design stability should be determined using low vegetative retardance conditions (Class D) and for design capacity higher

vegetative retardance conditions (Class C) should be used.
4. For vegetative channels, flow velocities within the channel should not exceed the maximum permissible velocities given in Tables 5-2 and 5-3.
5. If relocation of a stream channel is unavoidable, the cross-sectional shape, meander, pattern, roughness, sediment transport, and slope should conform to the existing conditions insofar as practicable. Some means of energy dissipation may be necessary when existing conditions cannot be duplicated.
6. Streambank stabilization should be provided, when appropriate, as a result of any stream disturbance such as encroachment and should include both upstream and downstream banks as well as the local site.
7. Open channel drainage systems are sized to handle a 25-year design storm. The 100-year design storm should be routed through the channel system to determine if the 100-year plus applicable building elevation restrictions are exceeded, structures are flooded, or flood damages increased.

5.2.2 Velocity Limitations

The final design of artificial open channels should be consistent with the velocity limitations for the selected channel lining. Maximum velocity values for selected lining categories are presented in Table 5-2. Seeding and mulch should only be used when the design value does not exceed the allowable value for bare soil. Velocity limitations for vegetative linings are reported in Table 5-3. Vegetative lining calculations are presented in Section 5.6 and riprap procedures are presented in Section 5.7.

5.3 Manning's n Values

The Manning's n value is an important variable in open channel flow computations. Variation in this variable can significantly affect discharge, depth, and velocity estimates. Since Manning's n values depend on many different physical characteristics of natural and man-made channels, care and good engineering judgment must be exercised in the selection process. Recommended Manning's n values for artificial channels with rigid, unlined, temporary, and riprap linings are given in Table 5-4. Recommended values for vegetative linings should be determined using Figure 5-1, which provides a graphical relationship between Manning's n values and the product of velocity and hydraulic radius for several vegetative retardance classifications (see Table 5-6). Figure 5-1 is used iteratively as described in Section 5.6. Recommended Manning's values for natural channels which are either excavated or dredged and natural are given in Table 5-5. For natural channels, Manning's n values should be estimated using the procedures presented in the publication Guide For Selecting Manning's Roughness Coefficients For Natural Channels And Flood Plains, FHWA-TS-84-204, 1984.

Table 5-2 Maximum Velocities for Comparing Lining Materials

Material Maximum Velocity (ft/s)
Sand
Silt
Firm Loam
Fine Gravel
Stiff Clay
Graded Loam or Silt to Cobbles
Coarse Gravel
Shales and Hard Pans
2.0
3.5
3.5
5.0
5.0

5.0
6.0
6.0

Table 5-3 Maximum Velocities for Vegetative Channel Linings

Vegetation Type Slope Range (%) 1 Maximum Velocity2 (ft/s)
Bermuda Grass
Bahia
Tall Fescue Grass
Mixtures 3
Kentucky Bluegrass
Buffalo Grass
Grass Mixture
Sericea Lespedeza,
Weeping Lovegrass
Alfalfa
Annuals5
Sod
Lapped sod
0 10
0 10
0-5
5-10
>10
0-51
5-10
0-54
0-5
5
4
4
6
4
4
4
3
3
4
5

1 Do not use on slopes steeper than 10 percent except for side-slope in combination channel.
2 Use velocities exceeding 5 ft/s only where good stands can be maintained.
3 Mixtures of Tall Fescue, Bahia, and/or Bermuda
4 Do not use on slopes steeper than 5 percent except for side slope in combination channel.
5 Annuals – used on mild slopes or as temporary protection until permanent covers are established.

5.4 Uniform Flow Calculations

5.4.1 Design Charts

Following is a discussion of the equations that can be used for the design and analysis of open channel flow. The Federal Highway Administration has prepared numerous design charts to aid in the design of rectangular, triangular, and trapezoidal open channel cross sections. In addition, design charts for grass-lined channels have been developed. These charts and instructions for their use are given in Section 5.10 of this chapter.

5.4.2 Manning's Equation

Manning's Equation, presented in three forms below, is recommended for evaluating uniform flow conditions in open channels:

v = (1. 49/ n) R 2/ 3 S 1/ 2 (5.1)

Q = (1. 49/ n) A R 2/ 3 S 1/ 2 (5.2)

S = [Qn/(1.49 A R 2/ 3)] 2 (5.3)

Where: v = average channel velocity (ft/ s)
Q = discharge rate for design conditions (cfs)
n = Manning's roughness coefficient
A = cross-sectional area (ft 2)
R = hydraulic radius A/ P (ft)
P = wetted perimeter (ft)
S = slope of the energy grade line (ft/ ft)

For prismatic channels, in the absence of backwater conditions, the slope of the energy grade line, water surface and channel bottom are equal.

5.4.3 Geometric Relationships

Area, wetted perimeter, hydraulic radius, and channel top width for standard channel cross-sections can be calculated from geometric dimensions. Irregular channel cross sections (i.e., those with a narrow deep main channel and a wide shallow overbank channel) must be subdivided into segments so that the flow can be computed separately for the main channel and overbank portions. This same process of subdivision may be used when different parts of the channel cross-section have different roughness coefficients. When computing the hydraulic radius of the subsections, the water depth common to the two adjacent subsections is not counted as wetted perimeter.

5.4.4 Direct Solutions

When the hydraulic radius, cross-sectional area, and roughness coefficient and slope are known, discharge can be calculated directly from equation 5. 2. The slope can be calculated using equation 5. 3 when the discharge, roughness coefficient, area, and hydraulic radius are known.

Nomographs for obtaining direct solutions to Manning's Equation are presented in Figures 5-2 and 5-3. Figure 5-2 provides a general solution for the velocity form of Manning's Equation, while Figure 5-3 provides a solution of Manning's Equation for trapezoidal channels.

Table 5-4 Manning's Roughness Coefficients For Artificial Channels -n

Depth Ranges
Category Lining Type 0-0.5 ft 0.5-2.0
ft
>2.0 ft
Rigid
Unlined
Temporary
Gravel Riprap
Rock Riprap
Concrete
Grouted Riprap
Stone Masonry
Soil Cement
Asphalt
Bare Soil
Rock Cut

Woven Paper Net
Jute Net
Fiberglass Roving
Straw With Net
Curled Wood Mat
Synthetic Mat
1-inch D50
2-inch D50
6-inch D50
12-inch D50
0.015
0.040
0.042
0.025
0.018
0.023
0.045
0.016
0.028
0.028
0.065
0.066
0.036
0.044
0.066
0.104
0.013
0.030
0.032
0.022
0.016
0.020
0.035
0.015
0.022
0.022
0.033
0.035
0.025
0.033
0.041
0.069
0.078
0.013
0.028
0.030
0.020
0.016
0.020
0.025
0.015

0.019
0.019
0.025
0.028
0.021
0.030
0.034
0.035
0.040
Note: Values listed are representative values for the respective depth ranges. Manning's roughness coefficients, n, vary
with the flow depth.
*Some "temporary" linings become permanent when buried. Source: HEC-15, 1988.

Table 5-5 Uniform Flow Values Of Roughness Coefficient -n
Type of Channel and Description Minimum Normal Maximum
EXCAVATED OR DREDGED
a. Earth, straight and uniform
1. Clean, recently completed
2. Clean, after weathering
3. Gravel, uniform section, clean
b. Earth, winding and sluggish
1. No vegetation
2. Grass, some weeds
3. Dense weeds/plants in deep channels
4. Earth bottom and drubble sides
5. Stony bottom and weedy sides
6. Cobble bottom and clean sides
c. Dragline-excavated or dredged
1. No vegetation
2. Light brush on banks
d. Rock cuts
1. Smooth and uniform
2. Jagged and irregular
e. Channels not maintained, weeds and brush uncut
1. Dense weeds, high as flow depth
2. Clean bottom, brush on sides
3. Same, highest stage of flow
4. Dense brush, high stage
NATURAL STREAMS
Minor streams (top width at floods stage < 100 ft)
a. Streams on Plain
1. Clean, straight, full stage, no rifts or deep
pools
2. Same as above, but more stones and weeds
3. Clean, winding, some pools and shoals
4. Same as above, but some weeds and some
stones
5. Same as above, lower stages, more
ineffective slopes and sections
6. Same as 4, but more stones

7. Sluggish reaches, deep pools, or floodways
with heavy stand of timber and underbrush
0.016
0.018
0.022
0.022
0.023
0.025
0.030
0.025
0.025
0.030
0.025
0.035
0.025
0.035
0.050
0.040
0.045
0.080
0.025
0.030
0.033
0.035
0.040
0.045
0.050
0.075
0.018
0.022
0.025
0.027
0.025
0.030
0.035
0.030
0.035
0.040
0.028
0.050
0.035
0.040
0.080
0.050
0.070
0.100
0.030
0.035
0.040
0.045
0.048

0.050
0.070
0.100
0.020
0.025
0.030
0.033
0.030
0.033
0.040
0.035
0.045
0.050
0.033
0.060
0.040
0.050
0.120
0.080
0.110
0.140
0.033
0.040
0.045
0.050
0.055
0.060
0.080
0.150

Table 5-5 Uniform Flow Values Of Roughness Coefficient – n (continued)

Type of Channel and Description Minimum Normal Maximum
b. Mountain streams, no vegetation, channel,
banks usually steep, trees and brush along banks
submerged at high stages
1. Bottom: gravels, cobbles, few boulders
2. Bottom: cobbles with large boulders
Floodplains
a. Pasture, no brush
1. Short grass
2. High grass
b. Cultivated area
1. No crop
2. Mature row crops
3. Mature field crops
c. Brush
1. Scattered brush, heavy weeds
2. Light brush and trees in winter
3. Light brush and trees in winter
4. Medium to dense brush, in winter
5. Medium to dense brush, in summer
c. Trees

1. Dense willows, summer, straight
2. Cleared land, tree stumps, no sprouts
3. Same as above, but with heavy growth of spouts
4. Heavy and of timber, a few down trees, little undergrowth, flood stage below branches
5. Same as above, but with flood stage reaching branches

Major Streams (top width at flood stage>100ft). Then value is less than that for minor streams of similar description, because banks offer less effective resistance.

a. Regular section with no boulders or brush
b. Irregular and rough section

0.030
0.040
0.025
0.030
0.020
0.025
0.030
0.035
0.035
0.040
0.045
0.070
0.110
0.030
0.050
0.080
0.100
0.025
0.035
0.040
0.050
0.030
0.035
0.030
0.035
0.040
0.050
0.050
0.060
0.070
0.100
0.0150
0.040
0.060
0.100
0.120

0.050
0.070
0.035
0.050
0.040
0.045
0.050
0.070
0.060
0.080
0.110
0.160
0.200
0.050
0.080
0.120
0.160
0.060
0.100

Table 5-6 Classification of Vegetal Covers as to Degrees of Retardance

Retardance Cover Condition
A
Weeping Lovegrass
Yellow Bluestem Ischaemum
Excellent stand, tall (average 30”)
Excellent stand, tall (average 36”)
B
Kudzu
Bermuda Grass
Native Grass Mixture
Little bluestem, bluestem,
blue gamma others short land
long stem Midwest lovegrass
Weeping love grass
Lespedeza sericea
Alfalfa
Weeping Lovegrass
Kudzu
Very dense growth, uncut
Good stand, tall (average 12”)
Good stand, unmowed
Good stand, tall (average 24”)
Good stand, not woody, tall (average 19”)
Good stand, uncut (average 11”)
Good stand, unmowed (average 13”)
Dense growth, uncut
C
Blue gamma
Crab Grass
Bermuda Grass

Common lespedeza
Grass-legume mixture
Summer (orchard grass
Redtop, Italian rye grass,
And common lespedeza)
Centipede Grass
Good stand, uncut (average 13”)
Fairstand, uncut (10-48”)
Good stand, mowed (average 6”)
Good stand, uncut (average 11”)
Good stand, uncut (6-8”)
Very dense cover (average 6”)
D
Kentucky bluegrass
Bermuda Grass
Common Lespedeza
Buffalo Grass
Grass-legume mixture:
Fall, spring (orchard
Grass, redtop, Italian Rye grass,
And common lespedeza)
Lespedeza Sericea
Good stand, headed (6-12”)
Good stand, cut to 2.5”
Excellent stand, uncut (average 4.5”)
Good stand, uncut (3-6”)
Good stand, uncut (4-5”)
After cutting to 2” (very good before
cutting)
E
Bermuda Grass
Bermuda Grass
Good stand, cut to 1.5”
Burned stubble
Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

General Solution Nomograph

The following steps are used for the general solution nomograph in Figure 5-2:

1. Determine open channel data, including slope in ft/ ft, hydraulic radius in ft, and Manning's n value.
2. Connect a line between the Manning's n scale and slope scale and note the point of intersection on the turning line.
3. Connect a line from the hydraulic radius to the point of intersection obtained in Step 2.
4. Extend the line from Step 3 to the velocity scale to obtain the velocity in ft/ s.

Trapezoidal Solution Nomograph

The trapezoidal channel nomograph solution to Manning's Equation in Figure 5-3 can be used to find the depth of flow if the design discharge is known or the design discharge if the depth of flow is known.

1. Determine input data, including slope in ft/ ft, Manning's n value, bottom width in ft, and side slope in ft/ ft.

2. a. Given the design discharge, find the product of Q times n, connect a line from the slope scale to the Qn scale, and find the point of intersection on the turning line.
b. Connect a line from the turning point from Step 2a to the b scale and find the intersection with the z = 0 scale.
c. Project horizontally from the point located in Step 2b to the appropriate z value and find the value of d/b.
d. Multiply the value of d/ b obtained in Step 2c by the bottom width b to find the depth of uniform flow, d.
3. a. Given the depth of flow, find the ratio d divided by b and project a horizontal line from the d/ b ratio at the appropriate side slope, z, to the z = 0 scale.
b. Connect a line from the point located in Step 3a to the b scale and find the intersection with the turning line.
c. Connect a line from the point located in Step 3b to the slope scale and find the intersection with the Qn scale.
d. Divide the value of Qn obtained in Step 3c by the n value to find the design discharge, Q.

5.4.5 Trial And Error Solutions

A trial and error procedure for solving Manning's Equation is used to compute the normal depth of flow in a uniform channel when the channel shape, slope, roughness, and design discharge are known. For purposes of the trial and error process, Manning's Equation can be arranged as:

$$AR^{2/3} = (Qn)/(1.49 S^{1/2}) \quad (5.4)$$

Where: A = cross-sectional area (ft)
R = hydraulic radius (ft)
Q = discharge rate for design conditions (cfs)
n = Manning’s roughness coefficient
S = slope of the energy grade line (ft/ ft)

To determine the normal depth of flow in a channel by the trial and error process, trial values of depth are used to determine A, P, and R for the given channel cross section. Trial values of $AR^{2/3}$ are computed until the equality of equation 5.4 is satisfied such that the design flow is conveyed for the slope and selected channel cross section. Graphical procedures for simplifying trial and error solutions are presented in Figure 5-4 for trapezoidal channels.

1. Determine input data, including design discharge, Q, Manning's n value, channel bottom width, b, channel slope, S, and channel side slope, z.
2. Calculate the trapezoidal conveyance factor using the equation:

$$K_T = (Qn)/(b^{8/3} S^{1/2}) \quad (5.5)$$

Where: K_T = trapezoidal open channel conveyance factor
Q = discharge rate for design conditions (cfs)
n = Manning’s roughness coefficient
b = bottom width (ft)
S = slope of the energy grade line (ft/ ft)

3. Enter the x-axis of Figure 5-4 with the value of K_T calculated in Step 2 and draw a line vertically to the curve corresponding to the appropriate z value from Step 1.
4. From the point of intersection obtained in Step 3, draw a horizontal line to the yaxis and read the value of the normal depth of flow over the bottom width, d/ b.
5. Multiply the d/ b value from Step 4 by b to obtain the normal depth of flow.

5.5 Critical Flow Calculations

5.5.1 Background

In the design of open channels, it is important to calculate the critical depth in order to determine if the flow in the channel will be subcritical or supercritical. If the flow is subcritical it is relatively easy to handle the flow through channel transitions because the flows are tranquil and wave action is minimal. In subcritical flow, the depth at any point is influenced by a downstream control, which may be either the critical depth or the water surface elevation in a pond or larger downstream channel. In supercritical flow, the depth of flow at any point is influenced by a control upstream, usually critical depth. In addition, the flows have relatively shallow depths and high velocities.
Critical depth depends only on the discharge rate and channel geometry. The general equation for determining critical depth is expressed as:

$Q^2/g = A^3/T$ (5.6)

Where: Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32. 2 ft/ sec^2)
A = cross-sectional area (ft^2)
T = top width of water surface (ft)
Note: A trial and error procedure is needed to solve equation 5-6.

5.5.2 Semi-Empirical Equations

Semi-empirical equations (as presented in Table 5-7) or section factors (as presented in Figure 5-5) can be used to simplify trial and error critical depth calculations. The following equation is used to determine critical depth with the critical flow section factor, Z:

$Z = Q/(g^{0.5})$ (5.7)

Where: Z = critical flow section factor
Q = discharge rate for design conditions (cfs)
g = acceleration due to gravity (32.2 ft/ sec^2)
The following guidelines are given for evaluating critical flow conditions of open channel flow:

1. A normal depth of uniform flow within about 10 percent of critical depth is unstable and should be avoided in design, if possible.
2. If the velocity head is less than one-half the mean depth of flow, the flow is subcritical.
3. If the velocity head is equal to one-half the mean depth of flow, the flow is critical.
4. If the velocity head is greater than one-half the mean depth of flow, the flow is supercritical.

The Froude number, Fr, calculated by the following equation, is useful for evaluating the type of flow conditions in an open channel:

$Fr = v/(gA/T)^{0.5}$ (5.8)

Where: Fr = Froude number (dimensionless)
v = velocity of flow (ft/ s)
g = acceleration of gravity (32. 2 ft/ sec^2)
A = cross-sectional area of flow (ft^2)
T = top width of flow (ft)
If Fr is greater than 1.0, flow is supercritical; if it is under 1.0, flow is subcritical. Fr is 1.0 for critical flow conditions.

Table 5-7

Critical Depth Equations For Uniform Flow In Selected Channel Cross Sections

Channel Type
Semi-Empirical Equation 2
for Estimating Critical Depth

Range of Applicability
1. Rectangular[3]
2. Trapezoidal[3]
3. Triangular[3]
4. Circular[4]
5. General[5]
$d_c = [Q^2/(gb^2)]^{1/3}$
$d_c = 0.81[Q^2/(gz^{0.75}b^{1.25})]^{0.27} - b/30z$
$d_c = [2Q^2)/(gz^2)]^{1/5}$
$d_c = 0.325(Q/D)^{2/3} + 0.083D$
$(A^3/T) = (Q^2/g)$
N/A
$0.1 < 0.5522Q/b^{2.5} < 0.4$
For $0.5522 Q/b^{2.5} < 0.1$, use
rectangular channels
equations
N/A
$0.3 < DC/d < 0.9$
N/A
Where: d_c = critical depth (ft)
Q = design discharge (cfs)
G = acceleration due to gravity (32.2 ft/s^2)
b = bottom width of channel (ft)
z = side slopes of channel (horizontal to vertical)
D = diameter of circular conduit (ft)
A = cross-sectional area of flow (ft^2)
T = top width of waters surface (ft)
[1] See Figure 5-5 for channel sketches
[2] Assumes uniform flow with the kinetic energy coefficient equal to 1.0
[3] Reference: French (1985)
[4] Reference: USDOT, FHWA, HDS-4 (1965)
[5] Reference: Brater and King (1976)

5.6 Vegetative Design

5.6.1 Introduction

A two-part procedure is recommended for final design of temporary and vegetative channel linings. Part 1, the design stability component, involves determining channel dimensions for low vegetative retardance conditions, using Class D as defined in Table 5-6. Part 2, the design capacity component, involves determining the depth increase necessary to maintain capacity for higher vegetative retardance conditions, using Class C as defined in Table 5-6. If temporary lining is to be used during construction, vegetative retardance Class E should be used for the design stability calculations.

If the channel slope exceeds 10 percent, or a combination of channel linings will be used, additional procedures not presented below are required. References include HEC15 (USDOT, FHWA, 1986) and HEC-14 (USDOT, FHWA, 1983).

5.6.2 Design Stability

The following are the steps for design stability calculations:

1. Determine appropriate design variables, including discharge, Q, bottom slope, S, cross-section parameters, and vegetation type.
2. Use Table 5-3 to assign a maximum velocity, v_m based on vegetation type and slope range.

3. Assume a value of n and determine the corresponding value of vR from the n versus vR curves in Figure 5-1. Use retardance Class D for permanent vegetation and E for temporary construction.
4. Calculate the hydraulic radius using the equation:
R = (vR)/ vm (5.9)
Where: R = hydraulic radius of flow (ft)
vR = value obtained from Figure 5-1 in Step 3
v m = maximum velocity from Step 2 (ft/ s)
5. Use the following form of Manning's Equation to calculate the value of vR:
vR = (1. 49 R 5/ 3 S 1/ 2)/ n (5.10)
Where: vR = calculated value of vR product
R = hydraulic radius value from Step 4 (ft)
S = channel bottom slope (ft/ ft)
n = Manning's n value assumed in Step 3
6. Compare the vR product value obtained in Step 5 to the value obtained from Figure 5-1 for the assumed n value in Step 3. If the values are not reasonably close, return to Step 3 and repeat the calculations using a new assumed n value.
7. For trapezoidal channels, find the flow depth using Figures 5-3 or 5-4, as described in Section 5.4.4. The depth of flow for other channel shapes can be evaluated using the trial and error procedure described in Section 5.5.
8. If bends are considered, calculate the length of downstream protection, Lp, for the bend using Figure 5-6. Provide additional protection, such as gravel or riprap in the bend and extending downstream for length, Lp.
5.6.3 Design Capacity
The following are the steps for design capacity calculations:
1. Assume a depth of flow greater than the value from Step 7 above and compute the waterway area and hydraulic radius (see Figure 5-5 for equations).
2. Divide the design flow rate, obtained using appropriate procedures from the Hydrology Chapter, by the waterway area from Step 1 to find the velocity.
3. Multiply the velocity from Step 2 by the hydraulic radius from Step 1 to find the value of vR.
4. Use Figure 5-1 to find a Manning's n value for retardance Class C based on the vR value from Step 3.
5. Use Manning's Equation (equation 5.1) or Figure 5-2 to find the velocity using the hydraulic radius from Step 1, Manning's n value from Step 4, and appropriate bottom slope.
6. Compare the velocity values from Steps 2 and 5. If the values are not reasonably close, return to Step 1 and repeat the calculations.
7. Add an appropriate freeboard to the final depth from Step 6. Generally, 20 percent is adequate.
8. If bends are considered, calculate superelevation of the water surface profile at the bend using the equation:
delta d = (v 2 T)/(gR c) (5.11)
Where: delta d = superelevation of the water surface profile due to the bend (ft)
v = average velocity from Step 6 (ft/ s)
T = top width of flow (ft)
g = acceleration of gravity (32. 2 ft/ sec2)
R c = mean radius of the bend (ft)
Note: Add freeboard consistent with the calculated delta d.
5.7 Riprap Design

5.7.1 Assumptions

The following procedure is based on results and analysis of laboratory and field data (Maynord, 1987; Reese, 1984; Reese, 1988). This procedure applies to riprap placement in both natural and prismatic channels and has the following assumptions and limitations:

1. Minimum riprap thickness equal to d_{100}
2. The value of d_{85}/d_{15} less than 4.6
3. Froude number less than 1.2
4. Side slopes up to 2:1
5. A safety factor of 1.2
6. Maximum velocity less than 18 feet per second

If significant turbulence is caused by boundary irregularities, such as installations near obstructions or structures, this procedure is not applicable.

5.7.2 Procedure

Following are the steps in the procedure for riprap design.

1. Determine the average velocity in the main channel for the design condition. Use the higher value of velocity calculated both with and without riprap in place (this may require iteration using procedures in Section 5.4.5). Manning's n values for riprap can be calculated from the equation:

$$\mathbf{n = 0.0395\,(d_{50})^{1/6} \quad (5.12)}$$

Where: n = Manning's roughness coefficient for stone riprap
d_{50} = diameter of stone for which 50 percent, by weight, of the gradation is finer (ft)

2. If rock is to be placed at the outside of a bend, multiply the velocity determined in Step 1 by the bend correction coefficient, C_b, given in Figure 5-7 for either a natural or prismatic channel. This requires determining the channel top width, T, just upstream from the bend and the centerline bend radius, R_b.
3. If the specific weight of the stone varies significantly from 165 pounds per cubic foot, multiply the velocity from Step 1 or 2 (as appropriate) by the specific weight correction coefficient, C_g, from Figure 5-8.
4. Determine the required minimum d_{30} value from Figure 5-9, based on the equation:

$$\mathbf{d_{30}/D = 0.193\,Fr^{2.5} \quad (5.13)}$$

Where: d_{30} = diameter of stone for which 30 percent, by weight, of the gradation is finer (ft)
D = depth of flow above stone (ft)
Fr = Froude number (see equation 5.8), dimensionless
v = mean velocity above the stone (ft/s)
g = acceleration of gravity (32. 2 ft/ sec^2)

5. Determine available riprap gradations. A well-graded riprap is preferable to uniform size or gap graded. The diameter of the largest stone, d_{100} , should not be more than 1.5 times the d_{50} size. Blanket thickness should be greater than or equal to d_{100} except as noted below. Sufficient fines (below d_{15}) should be available to fill the voids in the larger rock sizes. The stone weight for a selected stone size can be calculated from the equation:

$$\mathbf{W = 0.5236\,SW_s\,d^3 \quad (5.14)}$$

Where: W = stone weight (lbs)
d = selected stone diameter (ft)
SW_s = specific weight of stone (lbs/ ft^3)

Filter fabric or a filter stone layer should be used to prevent turbulence or groundwater seepage from removing bank material through the stone or to serve as a foundation for unconsolidated material. Layer thickness should be increased

by 50 percent for underwater placement.
6. If d_{85}/d_{15} is between 2.0 and 2.3 and a smaller d_{30} size is desired, a thickness greater than d_{100} can be used to offset the smaller d_{30} size. Figure 5-10 can be used to make an approximate adjustment using the ratio of d_{30} sizes. Enter the y-axis with the ratio of the desired d_{30} size to the standard d_{30} size and find the thickness ratio increase on the x-axis. Other minor gradation deficiencies may be compensated for by increasing the stone blanket thickness.
7. Perform preliminary design, ensuring that adequate transition is provided to natural materials both up and downstream to avoid flanking and that toe protection is provided to avoid riprap undermining.

5.8 Uniform Flow -Example Problems

Example 1 --Direct Solution of Manning's Equation

Use Manning's Equation to find the velocity, v, for an open channel with a hydraulic radius value of 0.6 ft, an n value of 0.020, and slope of 0.003 ft/ ft. Solve using Figure 5-2:

1. Connect a line between the slope scale at 0.003 and the roughness scale at 0.020 and note the intersection point on the turning line.
2. Connect a line between that intersection point and the hydraulic radius scale at 0.6 ft and read the velocity of 2.9 ft/ s from the velocity scale.

Example 2 --Grassed Channel Design Stability

A trapezoidal channel is required to carry 50 cfs at a bottom slope of 0.015 ft/ ft. Find the channel dimensions required for design stability criteria (retardance Class D) for a grass mixture.

1. From Table 5-3, the maximum velocity, v_m, for a grass mixture with a bottom slope less than 5 percent is 4 ft/ s.
2. Assume a n value of 0.035 and find the value of vR from Figure 5-1. vR = 5.
3. Use equation 5.9 to calculate the value of R: R = 5.4/ 4 = 1.35 ft
4. Use equation 5.10 to calculate the value of vR:
$vR = [1.49 (1.35)^{5/3} (0.015)^{1/2}]/0.035 = 8.60$
5. Since the vR value calculated in Step 4 is higher than the value obtained from Step 2, a higher n value is required and calculations are repeated. The results from each trial of calculations are presented below:

Assumed n value	vR (Figure 5-1)	R (eq. 5.9)	vR (eq. 5.10)
0.035	5.40	1.35	8.60
0.038	3.8	0.95	4.41
0.039	3.4	0.85	3.51
0.040	3.2	0.80	3.15

Select n = 0.040 for stability criteria.
6. Use Figure 5-3 to select channel dimensions for a trapezoidal shape with 3:1 side

slopes.
Qn = (50) (0.040) = 2.0
S = 0. 015
For b = 10 ft,
d = (10) (0.098) = 0.98 ft
b = 8 ft, d = (8) (0.14) = 1.12 ft
Select: b = 10 ft, such that R is approximately 0.80 ft
z = 3
d = 1 ft
v = 3.9 ft/ s (equation 5.1)
Fr = 0.76 (equation 5.8)
Flow is subcritical
Design capacity calculations for this channel are presented in Example 3 below.
Example 3 --Grassed Channel Design Capacity
Use a 10-ft bottom width and 3: 1 side-slopes for the trapezoidal channel sized in Example 2 and find the depth of flow for retardance Class C.
1. Assume a depth of 1. 0 ft and calculate the following (see Figure 5-5):
A = (b + zd) d = [10 + (3) (1)] (1) = 13.0 square ft
R = {[b+zd]d}/{b+ [2d(1+z 2) 0.5]} = {[10+(3)(1)](1)}/{10+[(2)(1)(1+3 2)0.5]}
R = 0. 796 ft
2. Find the velocity. v = Q/ A = 50/ 13.0 = 3.85 ft/ s
3. Find the value of vR. vR = (3. 85) (0.796) = 3.06
4. Using the vR product from Step 3, find Manning's n from Figure 5-1 for retardance Class C.
n = 0. 047
5. Use Figure 5-2 or equation 5.1 to find the velocity for
S = 0.015
R = 0.796
n = 0.047.
v = 3.34 ft/ s
6. Since 3.34 ft/ s is less than 3.85 ft/ s, a higher depth is required and calculations are repeated. Results from each trial of calculations are presented below:
Assumed
Depth (ft)
Area
(ft2) R (ft)
Velocity
Q/A
(ft/sec) vR
Manning’s
N
(Fig. 5-3)
Velocity
(Eq. 5.11)
1.0
1.05
1.1
1.2
13.00
13.81
14.63

16.32
0.796
0.830
0.863
0.928
3.85
3.62
3.42
3.06
3.06
3.00
2.95
2.84
0.047
0.047
0.048
0.049
3.34
3.39
3.45
3.54

7. Select a depth of 1.1 with an n value of 0. 048 for design capacity requirements. Add at least 0.2 ft for freeboard to give a design depth of 1.3 ft. Design data for the trapezoidal channel are summarized as follows:

Vegetation lining = grass mixture, vm= 4 ft/ s
Q = 50 cfs
b = 10 ft, d = 1. 3 ft, z = 3, S = 0. 015
Top width = (10) + (2) (3) (1.3) = 17.8 ft
n(stability) = 0.040, d = 1.0 ft, v = 3.9 ft/ s,
Froude number = 0.76 (equation 5.8)
n(capacity) = 0.048, d = 1.1 ft, v = 3.45 ft/ s,
Froude number = 0.64 (equation 5.8)

Example 4 --Riprap Design.

A natural channel has an average bankfull channel velocity of 8 ft per second with a top width of 20 ft and a bend radius of 50 ft. The depth over the toe of the outer bank is 5 ft. Available stone weight is 170 lbs/ ft 3 . Stone placement is on a side slope of 2: 1 (horizontal: vertical).

1. Use 8 ft per second as the design velocity, because the reach is short and the bend is not protected.
2. Determine the bend correction coefficient for the ratio of Rb/T = 50/20 = 2.5. From Figure 5-7, Cb = 1.55. The adjusted effective velocity is (8) (1.55) = 12.4 ft/s.
3. Determine the correction coefficient for the specific weight of 170 lbs from Figure 5-8 as 0.98. The adjusted effective velocity is (12.4) (0.98) = 12.15 ft/s.
4. Determine minimum d30 from Figure 5-9 or equation 5.13 as about 10 inches.
5. Use a gradation with a minimum d30 size of 12 inches that is acceptable. Usually those have enough fines that a filter course will not be required.
6. (Optional) Another gradation is available with a d30 of 8 inches. The ratio of desired to standard stone size is 8/ 10 or 0.8. From Figure 5-10, this gradation would be acceptable if the blanket thickness was increased from the original d100 (diameter of the largest stone) thickness by 35 percent (a ratio of 1.35 on the

horizontal axis).
7. Perform preliminary design. Make sure that the stone is carried up and downstream far enough to ensure stability of the channel and that the toe will not be undermined. The downstream length of protection for channel bends can be determined using Figure 5-6.

5.9 Gradually Varied Flow

The most common occurrence of gradually varied flow in storm drainage is the backwater created by culverts, storm sewer inlets, or channel constrictions. For these conditions, the flow depth will be greater than normal depth in the channel and the water surface profile should be computed using backwater techniques.

Many computer programs are available for computation of backwater curves. The most general and widely used programs are, HEC-2, developed by the U. S. Army Corps of Engineers (1982) and Bridge Waterways Analysis Model (WSPRO) developed for the Federal Highway Administration. These programs can be used to compute water surface profiles for both natural and artificial channels.

For prismatic channels, the backwater calculation can be computed manually using the direct step method. For an irregular non-uniform channel, the standard step method is recommended, although it is a more tedious and iterative process. The use of HEC-2 is recommended for standard step calculations.

Cross sections for water surface profile calculations should be normal to the direction of flood flow. The number of sections required will depend on the irregularity of the stream and flood plain. In general, a cross section should be obtained at each location where there are significant changes in stream width, shape, or vegetal patterns. Sections should usually be no more than 4 to 5 channel widths apart or 100 ft apart for ditches or streams and 500 ft apart for flood plains, unless the channel is very regular.

5.10 Rectangular, Triangular, and Trapezoidal Design Figures

5.10.1 Introduction

The Federal Highway Administration has prepared numerous design figures to aid in the design of open channels. Copies of these figures, a brief description of their use, and several example design problems are presented. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

5.10.2 Description Of Figures

Figures given in Appendix A, B, and C at the end of this chapter are for the direct solution of the Manning equation for various sized open channels with rectangular, triangular, and trapezoidal cross sections. Each figure (except for the triangular cross section) is prepared for a channel of given bottom width and a particular value of Manning's n.

The figures for rectangular and trapezoidal cross section channels (Appendix A) are used the same way. The abscissa scale of discharge in cubic feet per second (cfs), and the ordinate scale is velocity in feet per second (ft/s). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (ft), and slightly inclined lines representing channel slope (ft/ft). A heavy dashed line on each figure shows critical flow conditions. Auxiliary abscissa and ordinate scales are provided for use with other values of n and are explained in the example problems. In the figures, interpolations may be made not only on the ordinate and abscissa scales but also between the inclined lines representing depth and slope.

The chart for a triangular cross section in (Appendix B) is in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. The nomograph also may be used for shallow V-shaped sections by following the instructions on the chart.

5.10.3 Instructions For Rectangular And Trapezoidal Figures

Figures in Appendix A provide a solution of the Manning equation for flow in open channels of uniform slope, cross section, and roughness, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow.

For a given slope and channel cross section, when n is 0.015 for rectangular channels or 0.03 for trapezoidal channels, the depth and velocity of uniform flow may be read directly from the figure for that size channel. The initial step is to locate the intersection of a vertical line through the discharge (abscissa) and the appropriate slope line. At this intersection, the depth of flow is read from the depth lines, and the mean velocity is read on the ordinate scale.

The procedure is reversed to determine the discharge at a given depth of flow. Critical depth, slope, and velocity for a given discharge can be read on the appropriate scale at the intersection of the critical curve and a vertical line through the discharge.

Auxiliary scales, labeled Qn (abscissa) and Vn (ordinate), are provided so the figures can be used for values of n other than those for which the charts were basically prepared. To use these scales, multiply the discharge by the value of n and use the Qn and Vn scales instead of the Q and V scales, except for computation of critical depth or critical velocity. To obtain normal velocity V from a value on the Vn scale, divide the value by n. The following examples will illustrate these points.

Example Design Problem 1

Given: A rectangular concrete channel 5 ft wide with n = 0.015, .06 percent slope (S = .0006), discharging 60 cfs.

Find: Depth, velocity, and type of flow

Procedure: 1. From Appendix A select the rectangular figure for a 5 ft width (Figure 5-11).

2. From 60 cfs on the Q scale, move vertically to intersect the slope line S = 0.0006, and from the depth lines read dn = 3.7 ft.

3. Move horizontally from the same intersection and read the normal velocity, V = 3.2 ft/ s, on the ordinate scale.

4. The intersection lies below the critical curve, and the flow is therefore in the subcritical range.

Example Design Problem 2

Given: A trapezoidal channel with 2: 1 side slopes and a 4 ft bottom width, with n = 0.030, 0. 2 percent slope (S -0.002), discharging 50 cfs.

Find: Depth, velocity, type flow.

Procedure: 1. Select the trapezoidal figure for b = 4 ft.

2. From 50 cfs on the Q scale, move vertically to intersect the slope line S = 0.002 and from the depth lines read d n = 2.2 ft (Figure 5-12).

3. Move horizontally from the same intersection and read the normal velocity, V = 2.75 ft/ s, on the ordinate scale. The intersection lies below the critical curve, the flow is therefore subcritical.

Example Design Problem 3

Given: A rectangular cement rubble masonry channel 5 ft wide, with n = 0.025, .5 percent slope (S -0. 005), discharging 80 cfs.

Find: Depth velocity and type of flow

Procedure: 1. Select the rectangular figure for a 5 ft width (Figure 5-13).

2. Multiply Q by n to obtain Qn: 80 x 0. 025 = 2.0.

3. From 2.0 on the Qn scale, move vertically to intersect the slope line, S = 0.005, and at the intersection read dn = 3.1 ft.

4. Move horizontally from the intersection and read Vn = .13, then Vn/ n = 0.13/ 0.025 = 5.2 ft/ s.

5. Critical depth and critical velocity are independent of the value of

n so their values can be read at the intersection of the critical curve with a vertical line through the discharge. For 80 cfs, on Figure 5-13, dc = 2.0 ft and Vc = 7.9 ft/ s. The normal velocity, 5.2 ft/s (from step 4), is less than the critical velocity, and the flow is therefore subcritical. It will also be noted that the normal depth, 3.0 ft, is greater than the critical depth, 2.0 ft, which also indicates subcritical flow.

6. To determine the critical slope for Q = 80 cfs and n = 0. 025, start at the intersection of the critical curve and a vertical line through the discharge, Q = 80 cfs, finding dc (2.0 ft) at this point. Follow along this dc line to its intersection with a vertical line through Qn = 2.0 (step 2), at this intersection read the slope value Sc = 0. 015.

5.10.4 Grassed Channel Figures

The Manning equation can be used to determine the capacity of a grass-lines channel, but the value of n varies with the type of grass, development of the grass cover, depth, and velocity of flow. The variable value of n complicates the solution of the Manning equation. The depth and velocity of flow must be estimated and the Manning equation solved using the n value that corresponds to the estimated depth and velocity. The trial solution provides better estimates of the depth and velocity for a new value of n and the equation is again solved. The procedure is repeated until a depth is found that carries the design discharge.

To prevent excessive erosion, the velocity of flow in a grass-lined channel must be kept below some maximum value (referred to as permissible velocity). The permissible velocity in a grass-lined channel depends upon the type of grass, condition of the grass cover, texture of the soil comprising the channel bed, channel slope, and to some extent the size and shape of the drainage channel. To guard against overtopping, the channel capacity should be computed for taller grass than is expected to be maintained, while the velocity used to check the adequacy of the protection should be computed assuming a lower grass height than will likely be maintained.

To aid in the design of grassed channels the Federal Highway Administration has prepared numerous design figures. Copies of these figures are in Appendix C at the end of this chapter. Following is a brief description of general design criteria, instructions on how to use the figures, and several example design problems. For design conditions not covered by the figures, a trial-and-error solution of the Manning equation must be used.

5.10.5 Description Of Figures

The figures in Appendix C are designed for use in the direct solution of the Manning equation for various channel sections lined with grass. The figures are similar in appearance and use to those for trapezoidal cross sections described earlier. However, their construction is much more difficult because the roughness coefficient (n) changes as higher velocities and/ or greater depths change the condition of the grass. The effect of velocity and depth of flow on n is evaluated by the product of velocity and hydraulic radius V times R. The variation of Manning's n with the retardance (Table 5-6) and the product V times R is shown in Figure 5-1. As indicated in Table 5-6, retardance varies with the height of the grass and the condition of the stand. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices. Table 5-6 is used to determine retardance classification.

The grassed channel figures each have two graphs, the upper graph for retardance D and the lower graph for retardance C. The figures are plotted with discharge in cubic feet per second on the abscissa and slope in feet per foot on the ordinate. Both scales are logarithmic. Superimposed on the logarithmic grid are lines for velocity in feet per second and lines for depth in feet. A dashed line shows the position of critical flow.

5.10.6 Instructions For Grassed Channel Figures

The grassed channel figures provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section. The flow should not be affected by backwater and the channel should have length sufficient to establish uniform flow. The figures are sufficiently accurate for design of drainage channels of fairly uniform cross section and slope, but are not appropriate for irregular natural channels.

The design of grassed channels requires two operations: (1) selecting a section which has the capacity to carry the design discharge on the available slope and (2) checking the velocity in the channel to insure that the grass lining will not be eroded. Because the retardance of the channel is largely beyond the control of the designer, it is good practice to compute the channel capacity using retardance C and the velocity using retardance D. The calculated velocity should then be checked against the permissible velocities listed in Tables 5-2 and 5-3. The use of the figures is explained in the following steps:

(1) Select the channel cross-section to be used and find the appropriate figure.

(2) Enter the lower graph (for retardance C) on the figure with the design discharge value on the abscissa and move vertically to the value of the slope on the ordinate scale. As this intersection, read the normal velocity and normal depth and note the position of the critical curve. If the intersection point is below the critical curve, the flow is sub-critical if it is above, the flow is supercritical.

(3) To check the velocity developed against the permissible (Tables 5-2 and 5-3), enter the upper graph on the same figure and repeat step 2. Then compare the computed velocity with the velocity permissible for the type of grass, channel slope, and erosion resistance of the soil. If the computed velocity is less, the design is acceptable. If not, a different channel section must be selected and the process repeated.

Example Design Problem 1

Given: A trapezoidal channel in easily eroded soil, lined with a grass mixture with 4: 1 side slopes, and a 4 ft bottom width on slope of 0. 02 ft per foot (S=. 02), discharging 20 cfs.

Find: Depth, velocity, type of flow, and adequacy of grass to prevent erosion

Procedure: 1. From Appendix C select figure for 4:1 side slopes (Figure 5-14 on the next page).

2. Enter the lower graph with Q = 20 cfs, and move vertically to the line for S= 0.02. At this intersection read dn= 1.0 ft, and normal velocity Vn 2.6 ft/ s.

3. The velocity for checking the adequacy of the grass cover should be obtained from the upper graph, for retardance D. Using the same procedure as in step 2, the velocity is found to be 3.0 ft/ s. This is about three quarters of that listed as permissible, 4.0 ft/ s in Table 5-3.

Example Design Problem 2

Given: The channel and discharge of Example 1

Find: The maximum grade on which the 20 cfs could safely be carried

Procedure: 1. With an increase in slope (but still less than 5%), the allowable velocity is estimated to be 4 ft/ s (see Table 5-3). On the upper graph Figure 5-15 for short grass, the intersection of the 20 cfs line and the 4 ft/ s line indicates a slope of 3.7 percent and a depth of 0. 73 ft.

5.11 References

Chow, V. T., ed. 1959. Open Channel Hydraulics. McGraw Hill Book Co. New York.

French, R. H. 1985. Open Channel Hydraulics. McGraw Hill Book Co. New York.
Federal Highway Administration. 1989. Bridge Waterways Analysis Model (WSPRO), Users Manual, FHWA IP-89-027.
Harza Engineering Company. 1972. Storm Drainage Design Manual. Prepared for the Erie and Niagara Counties Regional Planning Bd. Harza Engineering Company, Grand Island, N. Y.
Maynord, S. T. 1987. Stable Riprap Size for Open Channel Flows. Ph. D. Dissertation. Colorado State University, Fort Collins, Colorado.
Morris, J. R. 1984. A Method of Estimating Floodway Setback Limits in Areas of Approximate Study. In Proceedings of 1984 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. Lexington, Kentucky: University of Kentucky.
Peterska, A. J. 1978. Hydraulic Design of Stilling Basins and Energy Dissipators. Engineering Monograph No. 25. U. S. Department of Interior, Bureau of Reclamation. Washington, D. C.
Reese, A. J. 1984. Riprap Sizing, Four Methods. In Proceedings of ASCE Conference on Water for Resource Development, Hydraulics Division, ASCE. David L. Schreiber, ed.
Reese, A. J. 1988. Nomographic Riprap Design. Miscellaneous Paper HL 88-2. Vicksburg, Mississippi: U. S. Army Engineers, Waterways Experiment Station.
U. S. Department of Transportation, Federal Highway Administration. 1973. Design Charts For Open Channel Flow. Hydraulic Design Series No. 3. Washington, D. C.
U. S. Department of Transportation, Federal Highway Administration. 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14. Washington, D. C.
U. S. Department of Transportation, Federal Highway Administration. 1984. Guide for Selecting Manning's Roughness Coefficients For Natural Channels and Flood Plains. FHWA-TS-84-204. Washington, D. C.
U. S. Department of Transportation, Federal Highway Administration. 1986. Design of Stable Channels with Flexible Linings. Hydraulic Engineering Circular No. 15. Washington, D. C.
Wright-McLaughlin Engineers. 1969. Urban Storm Drainage Criteria Manual, Vol. 2. Prepared for the Denver Regional Council of Governments. Wright-McLaughlin Engineers, Denver, Col.

6 Storage Facilities

6.1 Introduction
6.1.1 Overview

The traditional design of storm drainage systems has been to collect and convey storm runoff as rapidly as possible to a suitable location where it can be discharged. As areas urbanize this type of design may result in major drainage and flooding problems downstream. Under favorable conditions, the temporary storage of some of the storm runoff can decrease downstream flows and often the cost of the downstream conveyance system. Detention storage facilities can range from small facilities contained in parking lots or other on-site facilities to large lakes and reservoirs. This chapter provides general design criteria for detention/retention storage basins as well as procedures for performing preliminary and final sizing and reservoir routing calculations.

6.1.2 Location Considerations

It should be noted that the location of storage facilities is very important as it relates to the effectiveness of these facilities to control downstream flooding. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system that could decrease or increase flood peaks in different downstream locations. Thus it is important for the engineer to design storage facilities as a drainage structure that both controls runoff from a defined area and interacts with other drainage structures within the drainage basin. Effective stormwater management must be coordinated on a regional or basin-wide planning basis.

6.1.3 Detention And Retention

Urban stormwater storage facilities are often referred to as either detention or retention facilities. For the purpose of this chapter, detention facilities are those that are designed to reduce the peak discharge and only detain runoff for some short period of time. These facilities are designed to completely drain after the design storm has passed. Retention facilities are designed to contain a permanent pool of water. Since most of the design procedures are the same for detention and retention facilities, the term storage facilities will be used in this chapter to include detention and retention facilities. If special procedures are needed for detention or retention facilities these will be specified.

6.1.4 When to Use Storage Facilities

For each development over one and one-tenths (1.1) acres in size, a stormwater impact evaluation prepared by a registered professional engineer is required. If this evaluation indicates that a proposed development will increase runoff from the property to a level that cannot be accommodated within the downstream drainage system, then storage facilities may be used to control the runoff from the proposed development to a level that can be accommodated within the downstream drainage system.

6.2 Symbols and Definitions

To provide consistency within this chapter as well as throughout this manual, the following symbols will be used. These symbols were selected because of their wide use in technical publications. In some cases the same symbol is used in existing publications for more than one definition. Where they occur in this chapter, the symbol will be defined where it occurs in the test or equations.

Table 6-1 Symbols and Definitions

Symbol	Definition	Units
A	Cross sectional or surface area	ft2
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
$\tilde{t}$	Routing time period	sec
g	Acceleration due to gravity	ft/s2
H	Head on structure	ft
Hc	Height of weir crest above channel bottom	ft
I	Inflow rate	cfs
L	Length	ft
Q	Flow or outflow rate	cfs
S, Vs	Storage volume	ft3
t b	Time base on hydrograph	hrs
Ti	Duration of basin inflow	hrs
tp	Time to peak	hrs
Vs, S	Storage volume	ft3
W	Width of basin	ft
Z	Side slope factor	-

6.3 Design Criteria
6.3.1 General Criteria

An analysis of such storage facilities should consist of comparing the design flow at a point or points downstream of the proposed storage site with and without storage. In addition to the design flow, other flows in excess of the design flow that might be expected to pass through the storage facility should be included in the analysis (i.e., 100-year flood). The design criteria for storage facilities should include:

- Release rate,
- Storage volume,
- Grading and depth requirements,
- Safety considerations and landscaping,
- Outlet works, and location.

Note: The same hydrologic procedure shall be used to determine pre-and post-development hydrology.

6.3.2 Release Rate

Control structure release rates shall approximate pre-developed peak runoff rates for the 2-year through 25-year storms, with emergency overflow capable of handling the 100-year discharge. Design calculations are required to demonstrate that the facility will limit runoff from the 2-, 5-, 10-, and 25-year developed discharge rates to pre-developed peak discharge rates.

6.3.3 Storage

Storage volume shall be adequate to attenuate the post-development peak discharge rates to pre-developed discharge rates for the 2-year through 25-year storms. Routing calculations must be used to demonstrate that the storage volume is adequate. If sedimentation during construction causes loss of detention volume, design dimensions shall be restored before completion of the project. For detention basins, all detention volume shall be drained within 72 hours.

6.3.4 Grading and Depth

Following is a discussion of the general grading and depth criteria for storage facilities followed by criteria related to detention and retention facilities.

General

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Vegetated embankments shall be less than 20 feet in height and shall have side slopes no steeper than 3:1(horizontal to vertical). Riprap-protected embankments shall be no steeper than 2:1. Geotechnical slope stability analysis is recommended for embankments greater than 10 feet in height and is mandatory for embankment slopes steeper than those given above. Procedures for performing slope stability evaluation can be found in most soil engineering textbooks, including those by Spangler and Handy (1982) and Sowers and Sowers (1970).

A minimum freeboard of 1 foot above the 100-year design storm high water elevation shall be provided for impoundment depths of less than 20-feet. Impoundment depths greater than 20 feet are subject to the requirements of the Safe Dams Act unless the facility is excavated to this depth.

Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements, and required freeboard. Aesthetically pleasing features are also important in urbanizing areas.

Detention

Areas above the normal high water elevations of storage facilities should be sloped at a minimum of 5 percent toward the facilities to allow drainage and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff. The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 2 percent bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and present standing water conditions. Often a sediment collection forebay is provided with easy maintenance access.

Retention

The maximum depth of permanent storage facilities will be determined by site conditions. Design constraints, and environmental needs. In general, if the facility provides a permanent pool of water, a depth sufficient to discourage growth of weeds (without creating undue potential for anaerobic bottom conditions) should be considered. A depth of 6 to 8 feet is generally reasonable unless fishery requirements dictate otherwise. Aeration may be required impermanent pools to prevent anaerobic conditions. Where aquatic habitat is required, wildlife experts should be contacted for site-specific criteria relating to such things as depth, habitat, and bottom and shore geometry. In some cases a shallow bench along the perimeter is constructed to encourage emergent vegetation growth to enhance the pollution reduction capabilities or aesthetics of the pond.

6.3.5 Outlet Works

Outlet works selected for storage facilities typically include a principal spillway and an emergency overflow, and must be able to accomplish the design functions of the facility. Outlet works can take the form of combinations of drop inlets, pipes, weirs, and orifices. Slotted riser pipes are discouraged because of clogging problems, but curb openings may be used for parking lot storage. The principal spillway is intended to convey the design storm without allowing flow to enter an emergency outlet. For large storage facilities, selecting a flood magnitude for sizing the emergency outlet should be consistent with the potential threat to downstream life and property if the basin embankment were to fail. The minimum flood to be used to size the emergency outlet is the 100-year flood. The sizing of a particular outlet works shall be based on results of hydrologic routing calculations.

6.3.6 Location

In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. If several storage facilities are located within a particular basin it is important to determine what effects a particular facility may have on combined hydrographs in downstream locations. Multiple storage facilities located in the same drainage basin will affect the timing of the runoff through the conveyance system which could decrease or increase flood peaks in different downstream locations. Small facilities will only have minimal flood control benefits and these benefits will quickly diminish as the flood wave travels downstream.

The following procedure is recommended to determine downstream effects. For all proposed storage facilities, channel routing calculations should proceed downstream to a confluence point where the drainage area being analyzed represents ten percent of the total drainage area. At this point the effect of the hydrograph routed through the proposed storage facility on the downstream hydrograph can be assessed and shown not to have detrimental effects on downstream areas.

Detention can be located within floodplains and still effectively control flooding through the use of timing calculations. In this situation the flood peak coming down the stream rarely coincides with local on-site flooding. It is often advantageous to allow the on-site water to pass with simple erosion control and a properly sized conveyance system. Then locate the detention pond to .skim. the peak from the oncoming flood hydrograph through the use of a side-channel weir or a simple flow through depression along the banks.

6.3.7 Safe Dams Act

Under the dam safety act regulations a dam is an artificial barrier that does or may impound water and that is 20 feet or greater in height and has a maximum storage volume of 30 acre-feet or more. A number of exemptions are allowed from the Safe Dams Act and any questions concerning a specific design or application should be addressed to the Georgia Department of Natural Resources.

6.4 General Procedure

6.4.1 Data Needs

The following data will be needed to complete storage design and routing calculations.

- Inflow hydrograph for all selected design storms for fully developed and pre-developed conditions.
- Stage-storage curve for proposed storage facility (see Figure 6-1 below for an example).
- Stage-discharge curve for all outlet control structures (see Figure 6-2 below for an example).

Using these data a design procedure is used to route the inflow hydrograph through the storage facility with different basin and outlet geometry until the desired outflow hydrograph is achieved (see example 6.8).

6.4.2 Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. The data for this type of curve are usually developed using a topographic map and the double-end area frustum of a pyramid, prismoidal formulas or circular conic section. The double-end area formula is expressed as:

V1,2 = [(A1 + A2)/2]d (6.1)

Where: V1,2 = storage volume, ft3, between elevations 1 and 2
A1 = surface area at elevation 1, ft2
A2 = surface area at elevation 2, ft2
d = change in elevation between points 1 and 2, ft

The frustum of a pyramid is expressed as:

V = d/3 [A1 + (A1 x A,)0.5 + A2]/3 (6.2)

Where: V = volume of frustum of a pyramid, ft3
d = change in elevation between points 1 and 2, ft
A1 = surface area at elevation 1, ft2
A2 = surface area at elevation 2, ft2

The prismoisal formula for trapezoidal basins is expressed as:

V = LWD = (L = W) ZD2 = 4/3 Z2 D3 (6.3)

Where: V = volume of trapezoidal basin, ft3
L = length of basin at base, ft
W = width of basin at base, ft
D = depth of basin, ft
Z = side slope factor, ratio of horizontal to vertical

The circular conic section formula is:

V = 1.047 D (R1
2 = R2
2 = R1R2) (6.4)
V = 1.047 D (3R1
2 = 3ZDR1 = Z2D2) (6.5)

Where R1 and R2 = bottom and surface radii of the conic section, ft
D = depth of basin, ft
Z = side slope factor, ratio of horizontal to vertical

6.4.3 Stage Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage facility. A typical storage facility has two spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design flood without allowing flow to enter the emergency spillway. A pipe culvert, weir or other appropriate outlet can be used for the principal spillway of outlet. The emergency spillway is sized to provide a bypass for floodwater during a flood that exceeds the design capacity of the principal spillway. This spillway should be designed taking into account the potential threat to downstream life and property if the storage facility were to fail.

The stage-discharge curve should take into account the discharge characteristics of both the principal spillway and the emergency spillway.

6.4.4 Procedure

A general procedure for using the above data in the design of storage facilities is presented below.

1. Compute inflow hydrograph for runoff from the 2-, 5-, 10-, and 100-year design storms using the procedures outlined in the Hydrology Chapter. Both pre-and post-development hydrographs are required.

2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1 (see Section 6.7).

3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.

4. Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage.

5. Perform routing calculations using inflow hydrographs from Step 1 to check the preliminary design using the storage routing equations. If the routed post-development peak discharges from the 2-through 25-year design storms exceed the pre-development peak discharges, or if the peak stage varies significantly from the estimated peak stage from Step 4, then revise the estimated volume and return to Step 3.

6. Perform routing calculations using the 100-year hydrograph, for developed land use conditions, to determine if any increases in downstream flows from this hydrograph will cause damages and/or drainage and flooding problems. If problems will be created then the storage facility must be designed to control the increased flows from the 100-year storm. If not then consider emergency overflow from runoff due to the 100-year (or larger) design storm and established freeboard requirements.

7. Evaluate the downstream effects of detention outflows from all design storms to ensure that the routed hydrograph does not cause downstream flooding problems. The exit hydrograph from the storage facility should be routed through the downstream channel system until a confluence point is reached where the drainage area being analyzed represents 10 percent of the total drainage area.

8. Evaluate the control structure outlet velocity and provide channel and bank stabilization if the velocity will cause erosion problems downstream. See the Energy Dissipation Chapter for information on controlling outlet velocities and the design of energy dissipators.

This procedure can involve a significant number of reservoir routing calculations to obtain the desired results.

6.4.5 10 Percent Limit

The 10 percent limit procedure utilizes a hydrologic-hydraulic computer model to analyze the downstream effects of stormwater runoff from developments of different size, shape, physical characteristics, and location within larger drainage basins. Based on the model, the effects of a development process stabilizes at the point where the proposed development represents approximately 10 percent of the drainage area, depending on the size of the development and the amount of increase impervious area.

If the 10 percent analysis study shows that there is increased peak flows downstream, several alternatives are available to the engineer to deal with the increased flows. These alternatives include installing on-site detention facilities, using extended detention facilities, upgrading drainage structures or conveyance downstream, obtaining easements, controlling runoff from the development site with infiltration or methods other than detention, etc.

Detention facilities are required when they will provide positive benefits to the local drainage system and shall not be required when they will be ineffective or not needed, which shall be determined by the City of Loganville Stormwater Department.

The following basic steps for using the 10 percent downstream analysis include the following:

- Develop hydrographs for the design storms at the discharge point(s) from the proposed development. The proposed developed land use conditions within the development should be used to develop these hydrographs.
- Route these hydrographs through the downstream drainage system to a point downstream where the size of the proposed development represents 10 percent or less of the total drainage area that contributes runoff to this point. This point is called the 10 percent point.
- For all points of interest in the downstream drainage system, between the exit of the proposed development to the 10 percent point, develop hydrographs from the

contributing areas. Existing land use conditions should be used for this analysis for all areas not included in the proposed development. Points of interest would include locations where drainage from sub-watersheds intersect, known drainage and flooding problems exist, where structures might be affected by storm runoff, etc. As a minimum, hydrographs at the 10 percent point should be developed with and without the proposed development.

- A comparison of the routed hydrograph from the proposed development with the other downstream hydrographs should indicate whether or not the proposed development will increase downstream peak flows or have little or no effect on these peak flows.
- If major constrictions (e.g., storage facilities, undersized culverts) are present in the downstream analysis area that will affect the general characteristics of the hydrographs, the associated engineering parameters of these constrictions should be included in the analysis.
- In most cases general topographic maps, soils information, and a field check of the drainage system will provide the data needed for this analysis.
- Detailed survey information and backwater analysis should not be needed for most downstream analysis.

6.5 Outlet Hydraulics
6.5.1 Outlets

Sharp-crested weir flow equations for no end contractions, two end contractions, and submerged discharge conditions are presented below, followed by equations for broad-crested weirs, v-notch weirs, proportional weirs, and orifices, or combinations of these facilities. If culverts are used as outlets works, procedures presented in the Culvert Chapter should be used to develop stage-discharge data.

6.5.2 Sharp-Crested Weirs

A sharp-crested weir with no end contractions is illustrated below. This discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.4(H/H_c)] LH^{1.5} \qquad \textbf{(6.6)}$$

Where: Q = discharge, cfs
H = head above weir crest excluding velocity head, ft
Hc = height of weir crest above channel bottom, ft
L = horizontal weir length, ft

Figure 6-3
Sharp-Crested Weir No End Contractions

Figure 6-4
Sharp-Crested Weir And Head

A sharp-crested weir with two end contractions is illustrated below. The discharge equation for this configuration is (Chow, 1959):

$Q = [(3.27 + .04(H/H_c)] (L - 0.2H) H^{1.5}$ (6.7)

Where: Variables are the same as equation 6.4.

Figure 6-5
Sharp-Crested Weir, Two End Contractions

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$Q_s = Q_f(1 - (H_2/H_1)^{1.5})^{0.385}$ (6.8)

Where: Q_s = submergence flow, cfs
Q_f = free flow, cfs
H_1 = upstream head above crest, ft
H_2 = downstream head above crest, ft

6.5.3 Broad-Crest Weirs

The equation for the broad-crested weir is (Brater and King, 1976):

$Q = CLH^{1.5}$ (6.9)

Where: Q = discharge, cfs
C = broad-crested weir coefficient
L = broad-crested weir length, ft
H = head above weir crest, ft

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum C value of 3.087. For sharp corners on the broad-crested weir, a minimum C value of 2.6 should be used. Information on C values as a function of weir crest breadth and head is given in Table 6-2.

6.5.4 V-Notch Weirs

The discharge through a v-notch weir can be calculated from the following equation (Brater and King, 1976):

$Q = 2.5 \tan(\theta/2)H^{2.5}$ (6.10)

Where: Q = discharge, cfs
θ = angle of v-notch, degrees
H = head on apex of notch, ft

6.5.5 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharged relationship achieved by allowing the discharge area to vary nonlinearly with head.

Design equations for proportional weirs are (Sandvic, 1985):

$$Q = 4.97\, a^{0.5}\, b(H - a/3) \quad (6.11)$$
$$x/b = 1 - (1/3.17)\,(\arctan (y/a)^{0.5}) \quad (6.12)$$

Where: Q = discharge, cfs
Dimensions a, b, h, x, and y are shown below

6.5.6 Orifices

Pipes smaller that 12. may be analyzed as a submerged orifice if H/D is greater than 1.5. For square-edged entrance conditions,

$$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5} \quad (6.13)$$

Where: Q = discharge, cfs
A = cross-section area of pipe, ft^2
g = acceleration due to gravity, 32.2 ft/s^2
D = diameter of pipe, ft
H = head on pipe, from the center of pipe to the water surface

6.5.7 Combination Outlets

Combinations of weirs, pipes and orifices can be put together to provide a variable control stage-discharge curve suitable for control of multiple storm flows. They are generally of two types: shared outlet control and separate outlet controls. Shared outlet control is typically a number of individual outlet openings, weirs or drops at different elevations on a riser pipe or box that all flow to a common larger conduit or pipe. Separate outlet controls are less common and normally consist of a single opening through the dam of a detention facility in combination with an overflow spillway for emergency use. For a complete discussion of outlets and combination outlets see Municipal Stormwater Management by Debo and Reese.

Table 6-2 Broad Crested Weir Coefficient C Values As A Function of Weir Crest Breadth (b) and Head (H) Weir Crest Breadth (ft)

Measured Head, H_1 (ft)
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8

2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
0.50
2.80
2.92
3.08
3.30
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
3.32
0.75
2.75
2.80
2.89
3.04
3.14
3.20
3.26
3.29
3.32
3.31
3.32
3.32
3.32
3.32
3.32
3.32
3.32
1.00
2.69
2.72
2.75
2.85
2.98
3.08

3.20
3.28
3.31
3.30
3.31
3.32
3.32
3.32
3.32
3.32
3.32
1.50
2.62
2.64
2.64
2.68
2.75
2.86
2.92
3.07
3.07
3.03
3.28
3.32
3.32
3.32
3.32
3.32
3.32
2.00
2.54
2.61
2.61
2.60
2.66
2.70
2.77
2.89
2.88
2.85
3.07
3.20
3.32
3.32
3.32
3.32
3.32
2.50
2.48
2.60
2.60

2.60
2.64
2.65
2.68
2.75
2.74
2.76
2.89
3.05
3.19
3.32
3.32
3.32
3.32
3.00
2.44
2.58
2.68
2.67
2.65
2.64
2.64
2.68
2.68
2.27
2.81
2.92
2.97
3.07
3.32
3.32
3.32
4.00
2.38
2.54
2.69
2.68
2.67
2.67
2.65
2.66
2.66
2.68
2.72
2.73
2.76
2.79
2.88
3.07
3.32
5.00

25.37
2.50
2.70
2.68
2.68
2.66
2.65
2.65
2.65
2.65
2.67
2.66
2.68
2.70
2.74
2.79
2.88
10.00
2.49
2.56
2.70
2.69
2.68
2.69
2.67
2.64
2.64
2.64
2.64
2.64
2.64
2.64
2.64
2.64
2.64
15.00
2.68
2.70
2.70
2.64
2.63
2.64
2.64
2.63
2.63
2.63
2.63
2.63
2.63
2.63
2.63

2.63
2.63
1 Measured at least 2.5H upstream the weir.
Reference: Brater and King (1976).

6.6 Preliminary Detention Calculations
6.6.1 Storage Volume

For small drainage areas, a preliminary estimate of the storage volume required for peak flow attenuation may be obtained from a simplified design procedure that replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figures 6-7 shown below.

The required storage volume maybe estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_s = 0.5T_i(Q_i - Q_o) \quad (6.14)$$

Where: V_s = storage volume estimate, ft^3
Q_i = peak inflow rate, cfs
Q_o = Peak outflow rate, cfs
T_i = duration of basin inflow, sec

Any consistent units may be used for Equation 6.14

6.6.2 Alternative Method

An alternative preliminary estimate of the storage volume required for a specified peak flow reduction can be obtained by the following regression equation procedure (Wycoff & Singh, 1986).

1. Determine input data, including the allowable peak outflow rate Q_o, the peak flow rate of the inflow hydrograph, Q_i, the time base of the inflow hydrograph, t_b, and the time to peak of the inflow hydrograph, t_p.

1. Calculate a preliminary estimate of the ratio V_s/V_r, using the input data from Step 1 and the following equation:

$$V_s/V_r = [1.291(1-Q_o/ Q_i)^{0.753}]/[(t_b/t_p)^{0.411}] \quad (6.15)$$

Where: V_s = volume of storage, in
V_r = volume of runoff, in
Q_o = outflow peak flow, cfs
Q_i = inflow peak flow, cfs
t_b = time base of the inflow hydrograph, hr (Determined as time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
t_p = time to peak of the inflow hydrograph, hr

2. Multiply the peak flow rate of the inflow hydrograph, Q_i, times the potential peak flow reduction calculated in Step 2 to obtain the estimated peak outflow rate, Q_o, for the

selected storage volume.

6.6.3 Peak Flow Reduction

A preliminary estimate of the potential peak flow reduction for a selected storage volume can be obtained by the following procedure.

1. Determine volume of runoff, Vr, peak flow rate of the inflow hydrograph, Q1, time base of the inflow hydrograph, tb, time to peak of the inflow hydrograph tp, and storage volume, Vs.

2. Calculate a preliminary estimate of the potential peak flow reduction for the selected storage volume using the following equation (Singh, 1976):

Qo/Qi = 1 – 0.712(Vs/Vr)1.328(tb/tp)0.546 (6.16)

Where: Qo = outflow peak flow, cfs
Qi = inflow peak flow, cfs
Vs = volume of storage, in
Vr = volume of runoff, in
tb = time base of the inflow hydrograph, hr (Determined as the time from the beginning of rise to a point on the recession limb where the flow is 5 percent of the peak.)
tp = time to peak of the inflow hydrograph, in hours

3. Multiply the peak flow rate of the inflow hydrograph, Qi, times the potential peak flow reduction calculated from step 2 to obtain the estimated peak outflow rate, Qo, for the selected storage volume (see example 6.8.3).

6.7 Routing Calculations

The following procedure is used to perform routing through a reservoir or storage facility (Puls Method of storage routing).

1. Develop an inflow hydrograph, stage-discharge curve, and stage-storage curve for the proposed storage facility. Example stage-storage and stage-discharge curves are shown below.

2. Use the storage-discharge data from Step 1 to develop storage characteristics curves that provide values of S+(O/2)̃ t versus stage. An example tabulation of storage characteristics curve data is shown in Table 6-3.

3. For a given time interval, I1 and I2 are known. Given the depth of storage or stage, H1, at the beginning of that time interval, S1-(O1/2)̃ t can be determined from the appropriate storage characteristics curve (example given below).

4. Determine the value of S2 + (O2/2̃)t from the following equation:

S2 + (O2/2)̃ t = [S1-(O1/2)̃ t] + [(I1 + I2)2̃t] (6.17)

Where: S2 = storage volume at time 2, ft3

O2 = outflow rate at time 2, cfs
˜t = routing time period, sec
S1 = storage volume at time 1, ft3
O1 =outflow rate at time 1, cfs
I1 = inflow rate at time 1, cfs
I2 = inflow rate at time 2, cfs

Other consistent units are equally appropriate.

5. Enter the storage characteristics curve at the calculated value of S2 + (O2/2)˜t determined in Step 4 and read off a new depth of water, H2.

6. Determine the value of O2, which corresponds to a state of H2, determined in Step 5, using the stage-discharge curve.

7. Repeat Steps 1 through 6 by setting new values of I1, O1, S1, and H1 equal to the previous I2, O2, S2, and H2, and using a new I2 value. This process is continued until the entire inflow hydrograph ahs been routed through the storage basin.

6.8 Example Problem
6.8.1 Example

This example demonstrated the application of the methodology presented in this chapter for the design of a typical detention storage facility. Example inflow hydrographs and associated peak discharges for both pre-and post-development conditions are assumed to have been developed using hydrologic methods from the Hydrology Chapter. Note: In this example only the 5-and 25-year hydrographs are used. The 2-and 10-year hydrographs should also be checked to determine if the final design is adequate.

6.8.2 Design Discharge and Hydrographs

Storage facilities shall be designed for runoff from the 2-, 5-, 10-, 25-, and 50-year design storms and an analysis done using the 100-year design storm runoff to ensure that the structure can accommodate runoff from this storm without damaging adjacent and downstream property and structures. Example peak discharges from the 5-and 25-year design storm events are as follows:

- Pre-developed 5-year peak discharge = 150 cfs
- Pre-developed 25-year peak discharge = 200 cfs
- Post-development 5-year peak discharge = 190 cfs
- Post-development 25-peak discharge = 250 cfs

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 150 and 200 cfs for the 5-and 25-year storms, respectively.

Example runoff hydrographs are shown in Table 6-4 below. Inflow durations from the post-development hydrographs are about 1.2 and 1.25 hours, respectively, for runoff from the 5-and 25-year storms.

Table 6-4 Example Runoff Hydrographs

Pre-Development Runoff Post-Development Runoff

(1)	(2)	(3)	(4)	(5)
Time	5-Year	25-Year	5-Year	25-Year
(Hrs)	(cfs)	(cfs)	(cfs)	(cfs)
0	0	0	0	0
0.1	18	24	38	50
0.2	61	81	125	178
0.3	127	170	190>150	250>200
0.4	150	200	125	165
0.5	112	150	70	90
0.6	71	95	39	50
0.7	45	61	22	29
0.8	30	40	12	16
0.9	21	28	7	9
1.0	13	18	4	5
1.1	10	15	2	3
1.2	8	13	0	1

Finally, the 2-and 10-year hydrographs should then be routed through the storage facility to be sure these storms are adequately controlled.

6.8.3 Preliminary Volume Calculations

Preliminary estimates of required storage volumes are obtained using the simplified method outlines in Section 6.6. For runoff from the 2-and 10-year storms, the required storage volumes Vs, are computed using equation 6.14;

Vs = O.5Ti(Qi . Qo)

5-year storm: Vs = [O.5(1.2 x 3,600)(190 . 150)]/43,560 = 1.98 acre-feet

25-year storm: Vs . [O.5(1.25 x 3,600)(250 . 200)]/43,560 = 2.58 acre-ft.

6.8.4 Design and Routing Calculations

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 5-and 25-year design storms are presented below. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 5-and 25-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 3.1, a weir length of 4 feet, and no tailwater submergence. The capacity of storage relief structures was assumed to be negligible.

Table 6-5 Stage-Discharge-Storage Data

(1)	(2)	(3)	(4)	(5)
Stage	Q	S	S1 . (0/2)	S1 + (0/2)˜t
(ft)	(cfs)	(acre-feet)	(acre-feet)	(acre-feet)
0.0	0	0.00	0.00	0.00
0.9	10	0.26	0.30	0.22
1.4	20	0.42	0.50	0.33
1.8	30	0.56	0.68	0.43
2.2	40	0.69	0.85	0.52

2.5	50	0.81	1.02	0.60
2.9	60	0.93	1.18	0.68
3.2	70	1.05	1.34	0.76
3.5	80	1.17	1.50	0.84
3.7	90	1.28	1.66	0.92
4.0	100	1.40	1.81	0.99
4.5	120	1.63	2.13	1.14
4.8	130	1.75	2.29	1.21
5.0	140	1.87	2.44	1.29
5.3	150	1.98	2.60	1.36
5.5	160	2.10	2.76	1.44
5.7	170	2.22	2.92	1.52
6.0	180	2.34	3.08	1.60
6.4	200	2.58	3.41	1.76
6.8	220	2.83	3.74	1.92
7.0	230	2.95	3.90	2.00
7.4	250	3.21	4.24	2.17

Storage routing was conducted for runoff from both the 5-and 25-year design storms to confirm the preliminary storage volume estimates and to establish design water surface elevations. Routing results using the Stage-Discharge Data given above and the Storage Characteristics Curves given on Figures 6-8 and 6-9, and 0.1 hour time steps are shown below for runoff from the 5-and 25-year design storms, respectively. The preliminary design provides adequate peak discharge attenuation for both the 5-and 25-year design storms.

Since the routed peak discharge is lower than the maximum allowable peak discharges for both design storm events, the weir length could be increased or the storage decreased. If revisions are desired, routing calculations must be repeated.

Although not shown for this example, runoff from the 200-year storm should be routed through the storage facility and downstream to determine if structures or adjacent land areas will be damaged. If flood damages will result, the storage facility must then be designed to limit the runoff from the 100-year storm to undeveloped conditions. Also, the 100-year routed storm should be used to establish freeboard requirements and to evaluate emergency overflow and stability requirements. In addition, the preliminary design provides hydraulic details only. Final design should consider site constraints such as depth to water, side slope stability and maintenance, grading to prevent standing water, and provisions for public safety. Also, the 2- and 10-year storms should be checked.

6.9 Trash Racks and Safety Gates

Trash racks and safety grates serve several functions:

- they trap larger debris well away from the entrance to the outlet works where they will not clog the critical portions of the works;
- they trap debris in such a way that relatively easy removal is possible;
- they keep people and large animals out of confined conveyance and outlet areas;
- they provide a safety system whereby persons caught in them will be stopped prior to the very high velocity flows immediately at the entrance to outlet works and persons will be carried up and onto the outlet works allowing for an ability to climb to safety, and
- well designed trash racks can have an aesthetically pleasing appearance.

When designed well trash racks serve there purpose without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985, Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors including: head losses through the rack, structural convenience, safety, and size of outlet.

Trash racks at entrances to pipes and conduits should be sloped at 3H:1V to 5H:1V to allow trash to slide up the rack with flow pressure and rising water level, the slower the approach flow the flatter the angle. Rack opening rules-of-thumb abound in the literature. Figure 6-12 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g. a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack. Alternately debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978, UDFCD, 1991). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate. Nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations though many have difficult to estimate variables. For a discussion of head loss related to grates with example empirical loss equations see Debo & Reese, 1994.

6.10 References

Brater, E.F. and H.W. King. 1976. Handbook of Hydraulics. 6th ed. New York: McGrase Hill Book Company.

Chow, C.N. 1959. Open Channel Hydraulics. New York: McGraw Hill Book Company.

Debo, Thomas N. and Andrew J. Reese. 1994. Municipal Stormwater Management. Lewis Publishers: CRC Press, Inc., Boca Raton, Florida.

Sandvik, G.B. and G.F. Sowers. 1970. Introductory Soil Mechanics and Foundations. 3rd ed. New : York: MacMillan Publishing Company.

Spangler, M.G. and R.L. Handy. 1982. Soil Engineering. 4th ed. New York: Harper & Row.

Stormwater Management Manuel . Volume 2 Procedures. July 1988. Metropolitan Government of Nashville and Davidson County. The EDGe Group, Inc. and CH2M Hill.

Wycuff, R.L. and U.P. Singh. .Preliminary Hydrologic Design of Small Flood Detention Reservoirs. Water Resources Bulletin. Vol. 12, No. 2, pp 337-49.

7 Energy Dissipation

7.1 Symbols And Definitions

To provide consistency within this chapter as well as throughout this manual the following symbols will be used. These symbols were selected because of their wide use in many energy dissipation publications.

Symbol Definition Units

A
D
d50
dw
Fr
g
hs
L
La
LB
Ls
Pl
Q
Sv
t
tc
TW
VL
Vo
Vo
Vs
Wo
Ws
ye
yo
Cross section area
Height of box culvert
Size of riprap
Culvert width
Froude Number
Acceleration of gravity
Depth of dissipator pool
Length
Riprap apron length
Overall length of basin
Length of dissipator pool
Plasticity index
Rate of discharge
Saturated shear strength
Time of scour
Critical tractive shear stress
Tailwater depth
Velocity L feet from brink
Normal velocity at brink
Outlet mean velocity

Volume of dissipator pool
Diameter width of culvert
Width of dissipator pool
Hydraulic depth that brink
Normal flow depth that brink
Sq ft
Ft
Ft
Ft
-
ft/s2
ft
ft
ft
ft
ft
-
cfs
lbs/in2
min.
lbs/in2
ft
ft/s
ft/s
ft/s
ft2
ft
ft
ft
ft

7.2 Design Criteria

7.2.1 General Criteria

Energy Dissipators shall be employed whenever the velocity of flows leaving a stormwater management facility exceeds the erosion velocity of the downstream channel system.

7.2.2 Erosion Hazards

Erosion problems at culverts or the outlet from detention basins are common. Determination of the flow conditions, scour potential, and channel erosion resistance, shall be standard procedure for all designs. The only safe procedure is to design on the basis that erosion at a culvert outlet and the downstream channel is to be expected. Standard practice is to use the same headwall treatment at the culvert entrance and exit. It is important to recognize that the inlet is designed to improve culvert capacity or reduce head loss while the outlet structure should provide a smooth flow transition back to the natural channel or into an energy dissipator. Outlet structures should provide uniform redistribution or spreading of the flow without excessive separation and turbulence. Figure 7-1 on the next page provides the riprap size recommended for use downstream of energy dissipators.

7.2.3 Recommended Dissipators

For many designs, the following outlet protection and energy dissipators provide sufficient protection at a reasonable cost.

- Riprap apron

- Riprap outlet basins
- Baffled outlets

This chapter will focus on these measures. The reader is referred to the Federal Highway Administration Hydraulic Engineering Circular No. 14 entitled, Hydraulic Design Of Energy Dissipators For Culverts And Channels, for the design procedures of the other energy dissipators.

7.3 Design Procedure

1. If outlet protection is required, choose an appropriate type. Suggested outlet protection facilities and applicable flow conditions (based on Froude number and dissipation velocity) are described below. When outlet protection facilities are selected, appropriate design flow conditions and site-specific factors affecting erosion and scour potential, construction cost, and long-term durability should be considered.

Following is a discussion of applicable conditions for each outlet protection measure.

a. Riprap aprons may be used when the outlet Froude number (Fr) is less than or equal to 2.5. In general, riprap aprons prove economical for transitions from culverts to overland sheet flow at terminal outlets, but may also be used for transitions from culvert sections to stable channel sections. Stability of the surface at the termination of the apron should be considered.

b. Riprap outlet basins may also be used when the outlet Fr is less than or equal to 2.5. They are generally used for transitions from culverts to stable channels. Since riprap outlet basins function by creating a hydraulic jump to dissipate energy, performance is impacted by tailwater conditions.

c. Baffled outlets have been used with outlet velocities up to 50 feet per second. Practical application typically requires an outlet Froude number between 1 and 9. Baffled outlets may be used at both terminal outlet and channel outlet transitions. They function by dissipating energy through impact and turbulence and are not significantly affected by tailwater conditions.

2. If outlet protection is not provided, energy dissipation will occur through formation of a local scour hole. A cutoff wall will be needed at the discharge outlet to prevent structural undermining. The wall depth should be slightly greater than the computed scour hole depth, hs . The scour hole should then be stabilized. If the scour hole is of such size that it will present maintenance, safety, or aesthetic problems, other outlet protection will be needed.

3. Evaluate the downstream channel stability and provide appropriate erosion protection if channel degradation is expected to occur.

7.4 Riprap Aprons

7.4.1 Uses

A flat riprap apron can be used to prevent erosion at the transition from a pipe or box culvert outlet to a natural channel. Protection is provided primarily by having sufficient length and flare to dissipate energy by expanding the flow. Riprap aprons are appropriate when the culvert outlet Fr is less than or equal to 2.5.

7.4.2 Procedure

The procedure presented in this section is taken from USDA, SCS (1975). Two sets of curves, one for minimum and one for maximum tailwater conditions, are used to determine the apron size and the median riprap diameter, d50. If tailwater conditions are unknown, or if both minimum and maximum conditions may occur, the apron should be designed to meet criteria for both. Although the design curves are based on round pipes flowing full, they can be used for partially full pipes and box culverts. The design procedure consists of the following steps:

1. If possible, determine tailwater conditions for the channel. If tailwater is less than one-half the discharge flow depth (pipe diameter if flowing full), minimum tailwater conditions exist and the curves in Figure 7-2 apply. Otherwise, maximum tailwater conditions exist and the curves in Figure 7-3 should be used.
2. Determine the correct apron length and median riprap diameter, d50, using the appropriate curves from Figures 7-2 and 7-3. If tailwater conditions are uncertain, find the values for both minimum and maximum conditions and size the apron as shown in Figure 7-4.

a. For pipes flowing full:
Use the depth of flow, d, which equals the pipe diameter, in feet, and design discharge, in cfs, to obtain the apron length, La, and median riprap diameter, d50, from the appropriate curves.

b. For pipes flowing partially full: Use the depth of flow, d, in feet, and velocity, v, in feet/ second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d 50 , from the scale on the right. From the lower d and v intersection point, move vertically to the upper curves until intersecting the curve for the correct flow depth, d. Find the minimum apron length, La, from the scale on the left.

c. For box culverts: Use the depth of flow, d, in feet, and velocity, v, in feet/ second. On the lower portion of the appropriate figure, find the intersection of the d and v curves, then find the riprap median diameter, d_{50}, from the scale on the right. From the lower d and v intersection point, move vertically to the upper curve until intersecting the curve equal to the flow depth,

d. Find the minimum apron length, L_a, using the scale on the left.

1. If tailwater conditions are uncertain, the median riprap diameter should be the larger of the values for minimum and maximum conditions. The dimensions of the apron will be as shown in Figure 7-4. This will provide protection under either of the tailwater conditions.

7.4.3 Design Considerations

The following items should be considered during riprap apron design:

1. The maximum stone diameter should be 1.5 times the median riprap diameter. $d_{max} = 1.5 \times d_{50}$, d_{50} = the median stone size in a well-graded riprap apron.
2. The riprap thickness should be 1.5 times the maximum stone diameter or 6 inches, whichever is greater. Apron thickness = $1.5 \times d_{max}$ (Apron thickness may be reduced to $1.5 \times d_{50}$ when an appropriate filter fabric is used under the apron.)
3. The apron width at the discharge outlet should be at least equal to the pipe diameter or culvert width, dw. Riprap should extend up both sides of the apron and around the end of the pipe or culvert at the discharge outlet at a maximum slope of 2: 1 and a height not less than the pipe diameter or culvert height, and should taper to the flat surface at the end of the apron.
4. If there is a well-defined channel, the apron length should be extended as necessary so that the downstream apron width is equal to the channel width. The sidewalls of the channel should not be steeper than 2:1.
5. If the ground slope downstream of the apron is steep, channel erosion may occur. The apron should be extended as necessary until the slope is gentle enough to prevent further erosion.
6. The potential for vandalism should be considered if the rock is easy to carry. If vandalism is a possibility, the rock size must be increased or the rocks held in place using concrete or grout.

7.4.4 Example Designs

Example 7-1. Riprap Apron Design for Minimum Tailwater Conditions

A flow of 280 cfs discharges from a 66-inch pipe with a tailwater of 2 ft above the pipe invert. Find the required design dimensions for a riprap apron.

1. Minimum tailwater conditions = $0.5\ d_o$, d_o = 66 in = 5.5 ft therefore, $0.5\ d_o$ = 2.75 ft.
2. Since TW = 2 ft, use Figure 7-2 for minimum tailwater conditions.
3. By Figure 7-2, the apron length, La, and median stone size, d_{50}, are 38 ft and 1.2 ft, respectively.
4. The downstream apron width equals the apron length plus the pipe diameter: $W = d + L_a = 5.5 + 38 = 43.5$ ft
5. Maximum riprap diameter is 1.5 times the median stone size: $1.5 (d_{50}) = 1.5 (1.2) = 1.8$ ft
6. Riprap depth = $1.5 (d_{max}) = 1.5 (1.8) = 2.7$ ft.

Example 7-2. Riprap Apron Design for Maximum Tailwater Conditions

A concrete box culvert 5.5 ft high and 10 ft wide conveys a flow of 600 cfs at a depth of 5.0 ft. Tailwater depth is 5.0 ft above the culvert outlet invert. Find the design dimensions for a riprap apron.

1. Compute 0. 5 do = 0.5 (5.0) = 2.5 ft.
2. Since TW = 5.0 ft is greater than 2.5 ft, use Figure 7-3 for maximum tailwater conditions.
v = Q/ A = [600/(5) (10)] = 12 ft/ s
3. On Figure 7-3, at the intersection of the curve, do = 60 in and v = 12 ft/ s, d 50 = 0.4 foot. Reading up to the intersection with d = 60 in, find La = 40 ft.
4. Apron width downstream = d w + 0.4 L a = 10 + 0.4 (40) = 26 ft.
5. Maximum stone diameter = 1.5 d50 = 1.5 (0.4) = 0.6 ft.
6. Riprap depth = 1.5 dmax = 1.5 (0.6) = 0.9 ft.

7.5 Riprap Basin Design

7.5.1 Uses

One method to reduce the exit velocities from outlets is to install a riprap basin. A riprap outlet basin is a preshaped scour hole lined with riprap that functions as an energy dissipator by forming a hydraulic jump.

7.5.2 Basin Features

General details of the basin recommended in this chapter are shown on Figure 7-5. Principal features of the basin are:
1. The basin is preshaped and lined with riprap of median size (d50).
2. The floor of the riprap basin is constructed at an elevation of h s below the culvert invert. The dimension hs is the approximate depth of scour that would occur in a thick pad of riprap of size d50 if subjected to design discharge. The ratio of hs to d50 of the material should be between 2 and 4.
3. The length of the energy dissipating pool is 10 x hs or 3 x Wo whichever is larger. The overall length of the basin is 15 x hs or 4 x Wo whichever is larger.

7.5.3 Design Procedure

The following procedure should be used for the design of riprap basins.
1. Estimate the flow properties at the brink (outlet) of the culvert. Establish the outlet invert elevation such that TW/ yo < or = 0.75 for the design discharge.
2. For subcritical flow conditions (culvert set on mild or horizontal slope) utilize Figures 7-6 or 7-7 to obtain yo /D, then obtain Vo by dividing Q by the wetted area associated with yo . D is the height of a box culvert. If the culvert is on a steep slope, Vo will be the normal velocity obtained by using the Manning equation for appropriate slope, section, and discharge.
3. For channel protection, compute the Froude number for brink conditions with ye = (A/ 2) 1.5 . Select d 50 /ye appropriate for locally available riprap (usually the most satisfactory results will be obtained if 0.25 < d 50 /ye < 0.45). Obtain hs /ye from Figure 7-8, and check to see that 2 < hs /d50 < 4. Recycle computations if hs/d50 falls out of this range.

4. Size basin as shown in Figure 7-5.
5. Where allowable dissipator exit velocity is specified:
a. Determine the average normal flow depth in the natural channel for the design discharge.
b. Extend the length of the energy basin (if necessary) so that the width of the energy basin at section A-A, Figure 7-5, times the average normal flow depth in the natural channel is approximately equal to the design discharge divided by the specified exit velocity.
6. In the exit region of the basin, the walls and apron of the basin should be warped (or transitioned) so that the cross section of the basin at the exit conforms to the cross section of the natural channel. Abrupt transition of surfaces should be

avoided to minimize separation zones and resultant eddies.
7. If high tailwater is a possibility and erosion protection is necessary for the downstream channel, the following design procedure is suggested:
- Design a conventional basin for low tailwater conditions in accordance with the instructions above.
- Estimate centerline velocity at a series of downstream cross sections using the information shown in Figure 7-9.
- Shape downstream channel and size riprap using Figure 7-1 and the stream velocities obtained above.

Material, construction techniques, and design details for riprap should be in accordance with specifications in the Federal Highway publication HEC No. 11 entitled Use of Riprap For Bank Protection.

7.5.4 Design Considerations

Riprap basin design should include consideration of the following:
1. The dimensions of a scour hole in a basin constructed with angular rock can be approximately the same as the dimensions of a scour hole in a basin constructed of rounded material when rock size and other variables are similar.
2. When the ratio of tailwater depth to brink depth, TW/y_o, is less than 0.75 and the ratio of scour depth to size of riprap, h_s/d_{50}, is greater than 2.0, the scour hole should function very efficiently as an energy dissipator. The concentrated flow at the culvert brink plunges into the hole, a jump forms against the downstream extremity of the scour hole, and flow is generally well dispersed leaving the basin.
3. The mound of material formed on the bed downstream of the scour hole contributes to the dissipation of energy and reduces the size of the scour hole; that is, if the mound from a stable scoured basin is removed and the basin is again subjected to design flow, the scour hole will enlarge.
4. For high tailwater basins (TW/y_o greater than 0.75), the high velocity core of water emerging from the culvert retains its jet-like character as it passes through the basin and diffuses similarly to a concentrated jet diffusing in a large body of water. As a result, the scour hole is much shallower and generally longer. Consequently, riprap may be required for the channel downstream of the rocklined basin.
1. It should be recognized that there is a potential for limited degradation to the floor of the dissipator pool for rare event discharges. With the protection afforded by the 2(d_{50}) thickness of riprap, the heavy layer of riprap adjacent to the roadway prism, and the apron riprap in the downstream portion of the basin, such damages should be superficial.

6. See Standards in the in HEC No. 11 for details on riprap materials and use of filter fabric.
7. Stability of the surface at the outlet of a basin should be considered using the methods for open channel flow as outlined in the Open Channel Hydraulics Chapter of this manual.

7.5.5 Example Designs

Following are some example problems to illustrate the design procedures outlined.
Example No. 1
Given: Box culvert -8 ft by 6 ft Design Discharge Q = 800 cfs
Supercritical flow in culvert Normal flow depth = brink depth
Yo = 4 ft Tailwater depth TW = 2.8 ft
Find: Riprap basin dimensions for these conditions:
Solution: Definition of terms in steps 1-5 can be found in Figures 7-5 and 7-8.

1. yo = ye for rectangular section, therefore with yo given as 4 ft, ye = 4 ft.
2. Vo = Q/A = 800/(4 x 8) = 25 ft/ s
3. Froude Number = Fr = V/(g x ye) 0.5 (g = 32.2 ft/ s2)
Fr = 25/(32. 2 x 4) 0.5 = 2. 20 < 2.5 O. K.
4. TW/ ye = 2.8/ 4.0 = 0.7
Therefore TW/ ye < 0.75 O.K.
5. Try d50/ ye = 0.45, d50 = 0.45 x 4 = 1.80 ft
From Figure 7-8, hs/ ye = 1.6, hs = 4 x 1. 6 = 6. 4 ft
hs/ d50 = 6.4/ 1.8 = 3.6 ft, 2 < hs/ d50 < 4 O. K.
6. Ls = 10 x hs = 10 x 6.4 = 64 ft (Ls = length of energy dissipator pool)
Ls min = 3 x Wo = 3 x 8 = 24 ft, therefore use Ls = 64 ft
LB = 15 x hs= 15 x 6.4 = 96 ft (LB = overall length of riprap basin)
LB min = 4 x Wo = 4 x 8 = 32 ft, therefore use LB = 96 ft
7. Thickness of riprap: On the approach = 3 x d50 = 3 x 1.8 = 5.4 ft
Remainder = 2 x d50 = 2 x 1.8 = 3.6 ft
Other basin dimensions designed according to details shown in Figure 7-5.
Example No. 2
Given: Same design data as example problem number 1 except:
Tailwater depth TW = 4.2 ft
Downstream channel can tolerate only 7 ft/ s discharge
Find: Riprap basin dimensions for these conditions
Solutions: Note --High tailwater depth, TW/ yo = 4. 2/4 = 1.05 > 0.75
1. From example 1: d50 = 1.8 ft, hs = 6.4 ft, Ls = 64 ft, LB = 96 ft.
2. Design riprap for downstream channel. Utilize Figure 7-9 for estimating average velocity along the channel. Compute equivalent circular diameter De for brink area from:
A = 3. 14De 2 /4 = yo x Wo = 4 x 8 = 32 ft 2
De = ((32 x 4)/ 3. 14) 0.5 = 6. 4 ft
Vo = 25 ft/ s (From Example 1)
3. Set up the following table:
L/De L VL/Vo v1
Rock Size
D50 (ft)
(Assume)
(De = Wo)
10
15*
20
21
(Compute)
ft
64
96
128
135
(Fig. 7-9)
0.59
0.37
0.30
0.28
Ft/sec

14.7
9.0
7.5
7.0
(Fig. 7-1)
1.4
0.6
0.4
0.4
*L/Wo is on a logarithmic scale so interpolations must be logarithmically.
Riprap should be at least the size shown but can be larger. As a practical consideration, the channel can be lined with the same size rock used for the basin. Protection must extend at least 135 ft downstream from the culvert brink. Channel should be shaped and riprap should be installed in accordance with details shown in the HEC No. 11 publication.
Example No. 3
Given: 6 ft diameter CMC
Design discharge Q = 135 cfs
Slope channel So = 0.004
Manning's n = 0.024
Normal depth in pipe for Q = 135 cfs is 4.5 ft
Normal velocity is 5.9 ft/ s
Flow is subcritical
Tailwater depth TW = 2.0 ft
Find: Riprap basin dimensions for these conditions:
Solution:
1. Determine yo and Vo
From Figure 7-7, yo/ D = 0. 45
Q/ D 2.5 = 135/ 6 2.5 = 1. 53
TW/ D = 2 0/ 6 = 0. 33
yo = .45 x 6 = 2. 7 ft
TW/ yo = 2.0/ 2.7 = 0.74 TW/ yo < 0.75 O.K.
Determine Brink Area (A) for yo/ D = 0.45
From Uniform Flow in Circular Sections Table (from Culvert Chapter)
For yo/ D = d/ D = 0. 45
A/ D 2 = 0. 3428, therefore A = 0.3428 x 6 2 = 12.3 ft 2
Vo = Q/ A = 135/ 12. 3 = 11. 0 ft/ s
2. For Froude number calculations at brink conditions,
ye = (A/ 2) 1/ 2 = (12.3/ 2) 1/ 2 = 2.48 ft
3. Froude number = Fr = Vo/(32.2 x ye) 1/2 = 11/(32.2 x 2.48)1/2 = 1.23 < 2.5 O.K.
4. For most satisfactory results -0. 25 < d50/ ye < 0. 45
Try d50/ ye = 0.25
d50 = 0.25 x 2. 48 = 0.62 ft
From Figure 7-8, hs/ ye = 0.75, therefore hs = 0.75 x 2. 48 = 1.86 ft
Uniform Flow in Circular Sections Flowing Partly Full (From Culvert Chapter)
Check: hs/ d50 = 1.86/ 0.62 = 3, 2 < hs/ d50 < 4 O. K.
5. Ls = 10 x hs = 10 x 1. 86 = 18. 6 ft or Ls = 3 x Wo = 3 x 6 = 18 ft,
therefore use Ls = 18. 6 ft L B = 15 x hs = 15 x 1. 86 = 27. 9 ft or
LB = 4 x Wo = 4 x 6 = 24 ft, therefore use LB = 27. 9 ft
d50 = 0.62 ft or use d50 = 8 in
Other basin dimensions should be designed in accordance with details shown on Figure

7-5. Figure 7-10 is provided as a convenient form to organize and present the results of riprap basin designs. When using the design procedure outlined in this chapter, it is recognized that there is some chance of limited degradation of the floor of the dissipator pool for rare event discharges. With the protection afforded by the 3 x d50 thickness of riprap on the approach and the 2 x d50 thickness of riprap on the basin floor and the apron in the downstream portion of the basin, the damage should be superficial.

7.6 Baffled Outlets

7.6.1 Uses

The baffled outlet (also known as the Impact Basin -USBR Type VI) is a boxlike structure with a vertical hanging baffle and an end sill, as shown in Figure 7-11. Energy is dissipated primarily through the impact of the water striking the baffle and, to a lesser extent, through the resulting turbulence. This type of outlet protection has been used with outlet velocities up to 50 ft per second and with Froude numbers from 1 to 9. Tailwater depth is not required for adequate energy dissipation, but a tailwater will help smooth the outlet flow.

7.6.2 Design Procedure

The following design procedure is based on physical modeling studies summarized from the U. S. Department of Interior (1978). The dimensions of a baffled outlet as shown in Figure 7-11 should be calculated as follows:

1. Determine input parameters, including:

h = Energy head to be dissipated, in ft (can be approximated as the difference between channel invert elevations at the inlet and outlet)

Q = Design discharge, in cfs

v = Theoretical velocity, in ft/ s = 2gh

A = Q/ v = Flow area, in square ft

d = A 0.5 = Representative flow depth entering the basin, in ft (assumes square jet)

Fr = v/(gd) 0.5 = Froude number, dimensionless

2. Calculate the minimum basin width, W, in ft, using the following equation.

W/ d = 2.88Fr 0.566 or W = 2.88dFr 0.566

Where: W = minimum basin width, in ft

d = depth of incoming flow, in ft

Fr = v/(gd) 0.5 = Froude number, dimensionless

The limits of the W/ d ratio are from 3 to 10, which corresponds to Froude numbers 1 and 9. If the basin is much wider than W, flow will pass under the baffle and energy dissipation will not be effective.

3. Calculate the other basin dimensions as shown in Figure 7-11, as a function of W. Construction drawings for selected widths are available from the U. S. Department of the Interior (1978).

4. Calculate required protection for the transition from the baffled outlet to the natural channel based on the outlet width. A riprap apron should be added of width W, length W (or a 5-foot minimum), and depth f (W/ 6). The side slopes should be 1. 5: 1, and median rock diameter should be at least W/ 20.

5. Calculate the baffled outlet invert elevation based on expected tailwater. The maximum distance between expected tailwater elevation and the invert should be b + f or some flow will go over the baffle with no energy dissipation. If the tailwater is known and fairly controlled, the baffled outlet invert should be a distance, b/ 2 + f, below the calculated tailwater elevation. If tailwater is uncontrolled, the baffled outlet invert should be a distance, f, below the downstream channel invert.

6. Calculate the outlet pipe diameter entering the basin assuming a velocity of 12 ft per second flowing full.

7. If the entrance pipe slopes steeply downward, the outlet pipe should be turned horizontal for at least 3 ft before entering the baffled outlet.
8. If it is possible that both the upstream and downstream ends of the pipe will be submerged, provide an air vent of diameter approximately 1/ 6 the pipe diameter near the upstream end to prevent pressure fluctuations and possible surging flow conditions.

7.6.3 Example Design

A cross-drainage pipe structure has a design flow rate of 150 cfs, a head, h, of 15 ft from invert of pipe, and a tailwater depth, TW, of 3 ft above ground surface. Find the baffled outlet basin dimensions and inlet pipe requirements.

1. Compute the theoretical velocity from
$v = (2gh)^{0.5} = [2(32.2 \text{ ft/sec}^2)(15 \text{ ft})]^{0.5} = 31.1 \text{ ft/s}$
This is less than 50 ft/ s, so a baffled outlet is suitable.
2. Determine the flow area using the theoretical velocity as follows:
$A = Q/v = 150 \text{ cfs}/31.1 \text{ ft/sec} = 4.8$ square ft
3. Compute the flow depth using the area from Step 2.
$d = (A)^{0.5} = (4.8 \text{ ft}^2)^{0.5} = 2.12$ ft
4. Compute the Froude number using the results from Steps 1 and 3.
$Fr = v/(gd)^{0.5} = 31.1 \text{ ft/sec}/[(32.2 \text{ ft/sec}^2)(2.12 \text{ ft})]^{0.5} = 3.8$
5. Determine the basin width using equation 7.2 with the Froude number from Step 4.
$W = 2.88\, dFr^{0.566} = 2.88\,(2.12)(3.8)^{0.566} = 13.0$ ft (minimum)
Use 13 ft as the design width.
6. Compute the remaining basin dimensions (as shown in Figure 7-11):
L = 4/ 3 (W) = 17. 3 ft, use L = 17 ft, 4 in
f = 1/ 6 (W) = 2. 17 ft, use f = 2 ft, 2 in
e = 1/ 12 (W) = 1. 08 ft, use e = 1 foot, 1 in
H = 3/ 4 (W) = 9. 75 ft, use H = 9 ft, 9 in
a = 1/ 2 (W) = 6. 5 ft, use a = 6 ft, 6 in
b = 3/ 8 (W) = 4. 88 ft, use b = 4 ft, 11 in
c = 1/ 2 (W) = 6. 5 ft, use c = 6 ft, 6 in
Baffle opening dimensions would be calculated as shown in Figure 7-11.
7. Basin invert should be at b/ 2 + f below tailwater, or (4 ft, 11 in)/ 2 + 2 ft, 2 in = 4. 63 ft Use 4 ft 8 in; therefore, invert should be 2 ft, 8 in below ground surface.
8. The riprap transition from the baffled outlet to the natural channel should be 13 ft long by 13 ft wide by 2 ft, 2 in deep (W x W x f). Median rock diameter should be of diameter W/ 20, or about 8 in.
9. Inlet pipe diameter should be sized for an inlet velocity of about 12 ft/ s. $(3.14d)^2/4 = Q/v$; $d = [(4Q)/3.14v]^{0.5} = [(4(150 \text{ cfs})/3.14(12 \text{ ft/sec})]^{0.5} = 3.99$ ft
Use 48-inch pipe. If a vent is required, it should be about 1/ 6 of the pipe diameter or 8 in.

7.7 References

Federal Highway Administration. 1983. Hydraulic Design of Energy Dissipators for Culverts and Channels. Hydraulic Engineering Circular No. 14. Federal Highway Administration. 1967. Use of Riprap for Bank Protection. Hydraulic Engineering Circular No. 11.
Searcy, James K. 1967. Use of Riprap for Bank Protection. Federal Highway Administration. Washington, D. C.
U. S. Department of Interior, Bureau of Reclamation. 1978. Design of Small Canal Structures. Denver, Colorado.

8.1 Non-structural SMPs

8.1.1 Introduction

Stormwater management practices (SMPs) are the basic mitigation measures used in the stormwater quality management plans to control pollutants within the City. Section 8.2 of this chapter presents the details of structural stormwater management practices and their use within the municipal drainage system. The other major category of SMPs include the many non-structural or source control practices that can be used for pollution prevention and control of pollutants. In most cases it is much easier and less costly to prevent the pollutants from entering the drainage system than trying to control pollutants with structural SMPs. Thus within the .treatment train. concept, the non-structural SMPs should be the first line of defense in protecting the receiving streams. If used properly, the non-structural SMPs can be very effective in controlling pollutants and greatly reduce the need for structural SMPs. In addition, non-structural SMPs tend to be less costly, easier to design and implement and easier to maintain than structural SMPs. Non-structural SMPs normally do not have technical or engineering designs associated with them but are measures that the City or other agencies or groups might require or implement to assist in the management water quality and the control of pollutants within the City. Following is a brief discussion of some non-structural SMPs that can be used within a stormwater quality management plan for different portions of the City of Loganville Drainage System.

8.1.2 Public Education/ Participation

Public education/ participation is not so much a stormwater management practice as it is a method by which to implement SMPs. Public education/ participation are vital components of many of the individual source control SMPs. A public education and participation plan provides the City with a strategy for educating its employees, the public, and businesses about the importance of protecting stormwater from improper use, storage, and disposal of pollutants. City employees must be trained, especially those that work in departments not directly related to stormwater but whose actions affect stormwater. Residents must become aware that a variety of hazardous products are used in the home and that their improper use and disposal can pollute stormwater and groundwater supplies. Businesses, particularly smaller ones that may not be regulated by Federal, State, or local regulations, must be informed of ways to reduce their potential to pollute stormwater.

8.1.3 Land Use Planning/ Management

This SMP presents an important opportunity to reduce the pollutants in stormwater runoff by using a comprehensive planning process to control or prevent certain land use activities in areas where water quality is sensitive to development. It is applicable to all types of land use and represents one of the most effective pollution prevention practices. Subdivision regulations, zoning ordinances, preliminary plan reviews and detailed plan reviews, are tools that may be used to mitigate stormwater contamination in newly developing areas. Also, master planning, cluster development, terracing and buffers are ways to use land use planning as a SMP in the normal design for subdivisions and other urban developments. An impervious cover limitation is one of the more effective land use management tools, since nationwide research has consistently documented increases in pollution loads with increases in impervious cover. In

addition to controlling impervious area cover, directly connected impervious areas are kept to a minimum. This is especially
important for large impervious areas such as parking lots and highways and it can also be effective for small impervious areas such as roof drainage.

8.1.4 Material Use Controls

There are three major SMPs included in this category:

1. Housekeeping Practices
2. Safer Alternative Products
3. Pesticide/ Fertilizer Use

In housekeeping practices, the goal is to promote efficient and safe practices such as storage, use, cleanup, and disposal, when handling potentially harmful materials such as fertilizers, pesticides, cleaning solutions, paint products, automotive products, and swimming pool chemicals. In addition, the use of less harmful products can be promoted. Alternatives exist for most product classes including fertilizers, pesticides, cleaning solutions, and automotive and paint products.

Pesticides and fertilizers have become an important component of land use and maintenance for municipalities, commercial land uses and residential land owners. Any usage of pesticides and fertilizers increases the potential for stormwater pollution. SMPs for pesticides and fertilizers include education in their use, control runoff from affected areas, control times when they are used, provide proper disposal areas, etc.

8.1.5 Material Exposure Controls

There are two major SMPs included in this category:

1. Material Storage Control
2. Vehicle Use Reduction

Material storage control is used to prevent or reduce the discharge of pollutants to stormwater from material delivery and storage by minimizing the storage of hazardous materials onsite, storing materials in a designated area, installing secondary containment, conducting regular inspections, and training employees and subcontractors.

Vehicle use reduction is used to reduce the discharge of pollutants to stormwater from vehicle use by high-lighting the stormwater impacts, promoting the benefits to stormwater of alternative transportation, and integrating initiatives with existing or emerging regulations and programs.

8.1.6 Material Disposal And Recycling

There are three major SMPs included in this category:

1. Storm Drain System Signs
2. Household Hazardous Waste Collection
3. Used Oil Collection

Stenciling of the storm drain system (inlets, catch basins, channels, and creeks) with prohibitive language/ graphic icons discourages the illegal dumping of unwanted materials. Storm drain system signs act as highly visible source controls that are typically stenciled directly adjacent to storm drain inlets.

Household hazardous wastes are defined as waste materials which are typically found in homes or similar sources, which exhibit characteristics such as: corrosivity, ignitability, reactivity, and/ or toxicity, or are listed as hazardous materials by the EPA. Household hazardous waste collection programs are a preventative rather than curative measure and may reduce the need for more elaborate treatment controls. Programs can be a combination of permanent collection centers, mobile collection centers, curbside collection, recycling, reuse, and source reduction.

Used oil recycling is a responsible alternative to improper disposal practices such as dumping oil in the sanitary sewer or storm drain system, applying oil to roads for dust control, placing used oil and filters in the trash for disposal to landfill, or simply pouring used oil on the ground. Commonly used oil collection alternatives are a temporary .drop off. site on designated collection days or the use of private collectors such as automobile service stations, quick oil change centers and auto parts stores.

8.1.7 Spill Prevention And Cleanup

There are two major SMPs included in this category:

1. Vehicle Spill Control
2. Aboveground Tank Spill Control

The purpose of a vehicle spill control program is to prevent or reduce the discharge of pollutants to stormwater from vehicle leaks and spills by reducing the chance for spills by preventive maintenance, stopping the source of spills, containing and cleaning up spills, properly disposing of spill materials, and training employees. It is also very important to respond to spills quickly and effectively.

Aboveground tank spill control programs prevent or reduce the discharge of pollutants to stormwater by installing safeguards against accidental releases, installing secondary containment, conducting regular inspections, and training employees in standard operating procedures and spill cleanup techniques.

8.1.8 Dumping Controls

This SMP addresses the implementation of measures to detect, correct, and enforce against illegal dumping of pollutants on streets and into the storm drain system, streams, and creeks. Substances illegally dumped on streets and into the storm drain system and creeks include paints, used oil and other automotive fluids, construction debris, chemicals, fresh concrete, leaves, grass clippings, and pet wastes.

8.1.9 Connection Controls

There are three major SMPs included in this category:

1. Illicit Connection Prevention

2. Illicit Connection Detection and Removal
3. Leaking Sanitary Sewer Control

Illicit connection protection tries to prevent unwarranted physical connections to the storm drain system from sanitary sewers, floor drains, etc., through regulation, regular inspection, testing, and education. In addition, programs include implementation control procedures for detection and removal of illegal connections from the storm drain conveyance system. Procedures include field screening, follow-up testing, and complaint investigation.
Leaking sanitary sewer control includes implementing control procedures for identifying, repairing, and remediating infiltration, inflow, and wet weather overflows from sanitary sewers into the storm drain conveyance system. Procedures include field screening, testing, and com-plaint investigation.

8.1.10 Street/ Storm Drain Maintenance

There are seven major SMPs included in this category:

1. Roadway Cleaning
2. Catch Basin Cleaning
3. Vegetation Controls
4. Storm Drain Flushing
5. Roadway/ Bridge Maintenance
6. Detention/ Infiltration Device Maintenance
7. Drainage Channel/ Creek Maintenance

Roadway cleaning may help reduce the discharge of pollutants to stormwater from street surfaces by conducting cleaning on a regular basis. However, cleaning often removes the larger sizes of pollutants but not the smaller sizes. Most pollutants are deposited within three feet of the curb which is where the roadway cleaning should be concentrated. Catch basin cleaning on a regular basis also helps reduce pollutants in the storm drain system, reduces high pollutant concentrations during the first flush of storms, prevents clogging of the downstream conveyance system and restores the catch basins. sediment trapping capacity.

Vegetation control typically involves a combination of chemical (herbicide) application and mechanical methods. Mechanical vegetation control includes leaving existing vegetation, cutting less frequently, hand cutting, planting low maintenance vegetation, mulching, collecting and properly disposing of clippings and cuttings, and educating employees.

Storm drains can be .flushed. with water to suspend and remove deposited materials. Flushing is particularly beneficial for storm drain pipes with grades too flat to be self-cleansing and helps ensure pipes convey design flow and removes pollutants from the storm drain system. However, flushing will only push the pollutants into downstream receiving waters unless the discharge from the flushing is captured and removed from the drainage system. Jet-Vac trucks should be employed to remove debris from this process.

Roadway/ bridge maintenance is used to prevent or reduce the discharge of pollutants to stormwater by paving as little as possible, designing bridges to collect and convey stormwater to proper locations, using measure to prevent runoff from entering the drainage system, properly disposing of maintenance wastes, and training employees. Proper maintenance and siltation removal is required on both a routine and corrective basis to promote effective stormwater pollutant removal efficiency for wet and dry detention ponds and infiltration devices. Also, regularly removing illegally dumped items and material from storm drainage channels and creeks will reduce pollutant levels.

8.1.11 Permanent Erosion Control

There are three major SMPs included in this category:

1. Erosion Control -Permanent Vegetation
2. Erosion Control -Flow Control
3. Erosion Control -Channel Stabilization

Vegetation is a highly effective method for providing long term, cost effective erosion protection for a wide variety of conditions. It is primarily used to protect the soil surface from the impact of rain and the energy of the wind. Vegetation is also effective in reducing the velocity and sediment load in runoff sheet flow.

Channel stabilization addresses the problem of erosion due to concentrated flows. Concentrated flows occur in channels, swales, creeks, rivers and other watercourses in which a substantial drainage area drains into a central point. Overland sheet flow begins to collect and concentrate in the form of rills and gullies after overland flow of as little as 100 feet. Erosion due to concentrated flow is typically extensive, causing large soil loss, undermining foundations and decreasing the flow capacity of watercourses.

Proper selection of ground cover is dependent on the type of soil, the time of year of planting, and the anticipated conditions that the ground cover will be subjected. In addition, mulching is a form of erosion protection that is commonly used in conjunction with establishment of vegetation. It typically improves infiltration of water, reduces, runoff, holds seed, fertilizer and lime in place, retains soil moisture, helps maintain temperatures, aids in germination, retards erosion and helps establish plants in disturbed areas.

Once flow is allowed to concentrate, it is more difficult to control erosion problems. Thus every effort should be made to maintain sheet flow conditions for runoff. Where concentrated flows are unavoidable, the following techniques can be used to control erosion and resulting water quality problems:

- Rip Rap
- Gabions
- Check Dams
- Level Spreaders
- Armor Protection
- Diversions

For more information on erosion control consult the publication, Manual For Erosion and Sediment Control in Georgia, available from the Georgia Soil and Water Conservation Commission.

8.2 Structural SMP Specifications

8.2.1 Introduction

To provide some guidance in the design and use of different structural SMP.s this section gives specifications and performance standards for several SMP.s that could find application within the City.

Following are the required specifications, recommended specifications, operation and maintenance requirements, and performance standards for nine different structural SMP.s. For the design of extended detention ponds and retention ponds refer to the storage chapter for examples of storage design. For grassed swales refer to the channels chapter for examples of channel design. For example designs of the several infiltration facilities included as SMPs in the chapter, refer to Appendix A at the end of this chapter.

- Extended Detention Ponds
- Retention (Wet) Ponds
- Constructed Wetlands
- Infiltration Trenches
- Filter Strips and Flow Spreaders
- Sand Filters
- Grassed Swales
- Oil/ Grit Separators

For each structural SMP, performance standards are included to give a general idea of the pollution removal rates of different structural SMPs. The general design criteria for the City of Loganville is to design for the runoff from the first inch of rainfall, from the entire drainage area above the SMP, with a detention time of 48 hours. However, the literature related to performance standards often gives removal rates for different design criteria (e. g., 24 hour detention time, 0.5 inch per impervious area). Thus these values should only be used for comparison of different structural SMPs and not for specific designs or to state accurate removal rates for different SMPs. Note that with the addition of appropriate vegetation, extended detention and retention ponds can function as bioretention facilities.

8.2.2 Extended Detention Ponds

Standard Specifications For Extended Detention Ponds

Required Specifications

Extended detention ponds shall be designed with a detention time of 48 hours. If the extended detention pond is to be designed for only water quality purposes, then the pond should be designed to capture the first 1.2 inches of runoff for the entire drainage area above the facility.

- Pilot channel of paved or concrete material for erosion control (alternately use turf if there is little low flow). Size such that any event runoff will overflow the low flow channel onto the pond floor.
- Side slopes shall be no greater than 3: 1 if mowed.
- Inlet and outlet located to maximize flow length.
- Design for full development upstream of control.
- Rip-rap protection (or other suitable erosion control means) for the outlet and all inlet structures into the pond.

- One and one-half (1 l/ 2) foot minimum freeboard above peak stage for top of embankment.
- Emergency spillway designed to pass the 100-year storm event (must be paved in fill areas).
- Maintenance access minimum of 25 feet wide.
- Trash racks, filters or other debris protection on control.
- Anti-vortex plates.
- Insure no outlet leakage and use anti-seep collars.
- Benchmark for sediment removal.

Recommended Specifications

- Two stage design (top stage -dry during the 1 inch rainfall event, bottom stage - inundated during storms equal to or less than the 1 inch storm event.)
- Top stage shall have slopes between 2% and 5% and a depth of 2 to 5 feet.
- Bottom stage maintained as shallow wetland or pool (6 to 12 in.).
- Manage buffer and pond as meadow.
- Minimum 25-foot wide buffer around pool.
- On-site disposal areas for two sediment removal cycles.
- Anti-seep collars on barrel of principal spillway.
- Impervious soil boundary.
- Design as off-line pond to bypass larger flows.
- Design as sediment settling basin for pretreatment of the larger particles.

Operation And Maintenance Recommendations

A stormwater management easement and maintenance agreement shall be required for each facility.

- Extended dry ponds are used where lack of water or other multi-use considerations preclude the use of wet ponds or constructed wetlands.
- Operation and maintenance is the same as for detention ponds (see storage chapter).
- Maintenance activities include keeping the outlets unclogged, controlling of vegetation, removing sediment deposits, and keeping aesthetics of area acceptable.

Performance Standards

- Soluble pollutant removal rates are low for extended dry detention ponds but can be enhanced either with greatly increased detention time, through the use of shallow marshes to increase biological uptake, or through using an infiltration device downstream from the outlet orifice.
- Average annual pollutant removal capability of extended detention ponds are as follows:

Pollutant 1 Inch Rain
Detained 24 hours

Same as Previous
W/ Shallow Marsh
Sediment
Total Phosphorus
Total Nitrogen
BOD
Metals
80-100%
40-60%
20-40%
40-60%
60-80%
80-100%
60-80%
40-60%
40-60%
60-80%

8.2.3 Retention Ponds

Standard Specifications For Retention (Wet) Ponds

Required Specifications

Retention ponds shall be designed with a minimum detention time of 48 hours. If the retention pond is to be designed for only water quality purposes, then the pond should be designed to capture the first 1.5-inch of runoff for the entire drainage area above the facility.

- Minimum length to width ratio of 3: l (preferably expanding outward toward the outlet). Irregular shorelines for larger ponds provide visual variety.
- Inlet and outlet located to maximize flow length. Use baffles if short circuiting cannot be prevented with inlet-outlet placement.
- Minimum depth of permanent pool 2 to 3 feet, maximum depth of 9 to 12 feet. Average depth should be 3 to 7 feet.
- Design for full development upstream of control.
- Side slopes shall be no greater than 3: l if mowed.
- Rip-rap protection (or other suitable erosion control means) for the outlet and all inlet structures into the pond. Individual boulders or baffle plates can work for this.
- Minimum drainage area of 20 acres.
- Anti-seep collars on barrel of principal spillway.
- One and one-half (1 l/ 2) foot minimum freeboard above peak stage for top of embankment.
- Emergency drain; i.e. sluice gate, drawdown pipe; capable of draining within 24 hours.
- Emergency spillway designed to pass the l00-year storm event.
- Bypass greater than the design storms.
- Trash racks, filters, hoods or other debris control on riser.
- Maintenance access minimum of 25 feet wide.
- Benchmark for sediment removal.

- Paved or concrete channel.
- Emergency drain to allow draw-down within 24 hours.
- The ratio of permanent pool storage to mean storm runoff shall greater than or equal to 4.

Recommended Specifications

- Multi-objective use such as amenities or flood control.
- Landscaping management of buffer as meadow.
- Design for multi-function as flood control and extended detention.
- Minimum length to width ratio of 3: 1 to 4: 1 (preferably wedge shaped).
- Use reinforced concrete instead of corrugated metal.
- Sediment forebay for larger ponds (often designed for 5 to 15 percent of total volume). Forebay should have separate drain for de-watering. Grass biofilters for smaller ponds.
- Consider artificial mixing for small sheltered ponds.
- Provision shall be made for vehicle access at a 4: 1 slope.
- Impervious soil boundary to prevent drawdown.
- Shallow marsh area around fringe 25 to 50 percent of area (including aquatic vegetation).
- A safety bench with a minimum width of 10 feet should be provided around the permanent pool.
- Minimum 25-foot wide buffer around pool.
- On-site disposal areas, for two sediment removal cycles, protected from runoff.
- An oil and grease skimmer for sites with high production of such pollutants.

Operation And Maintenance Recommendations

- Sediment to be removed when 20% of storage volume of the facility is filled (design storage volume must account for volume lost to sediment storage).
- Sediment traps shall be cleaned out when filled.
- No woody vegetation shall be allowed on the embankment without special design provisions.
- Other vegetation over 18 inches high shall be cut unless it is part of planned landscaping.
- Debris shall be removed from blocking inlet and outlet structures and from areas of potential clogging.
- The control shall be kept structurally sound, free from erosion, and functioning as designed.
- Periodic removal of dead vegetation shall be accomplished.
- No standing water is allowed within extended detention pond unless specifically designed for.
- Inspection requirements should he outlined in the maintenance agreement.
- The site should be inspected and debris removed after every major storm.
- All special maintenance responsibilities will be listed in the maintenance agreement.
- Mow embankment and side slopes at least twice a year.
- Consider chemical treatment by alum if algal blooms are a problem.

Performance Standards

- Wet ponds are very effective in removal of both the soluble and particulate fractions of pollution.
- Average annual pollutant removal capabilities of wet ponds are as follows.

Pollutant

Sediment
Total Phosphorus
Total Nitrogen
BOD
Metals
0.5 Inch Per Impervious Acre

60-80%
40-60%
20-40%
20-40%
20-40%

8.2.4 Sand Filters

Standard Specifications For Sand Filters

Required Specifications

- Maximum contributing drainage area of 50 acres.
- 1-inch rainfall design storm.
- Designed to completely empty in 48 hours or more.
- Inlet structure shall be designed to spread the flow uniformly across the surface of the filter media.
- Stone riprap or other dissipation devices shall be installed to prevent gouging of the sand media and to promote uniform flow.
- Final sand bed depth shall be at least 18 inches.
- Underdrain pipes shall consist of main collector pipes and perforated lateral branch pipes.
- The underdrain piping shall be reinforced to withstand the weight of the overburden.
- Internal diameters of lateral branch pipes shall be 4 inches or greater and perforations shall be 3/8 inch.
- Maximum spacing between rows of perforations shall not exceed 6 inches.
- All piping shall be schedule 40 polyvinyl chloride or greater strength.
- Minimum grade of piping shall be l/ 8 inch per foot (1% slope).
- Access for cleaning all underdrain piping shall be provided.

A minimum infiltration rate of 0.5 inches per hour should be used for all infiltration designs, including sand filters.

Recommended Specification

Two sand filter configurations are recommended for use:

1) Sand Bed with Gravel Layer;
o Top layer of sand shall be a minimum of 18 inches of 0.02 -0.04-inch diameter sand (smaller sand size is acceptable).
o A layer of one-half to 2-inch diameter gravel under the sand shall be provided for a minimum of 2 inches of cover over the top of the under-drain lateral pipes.
o No gravel is required under the lateral pipes.
o The sand and gravel shall be separated by a layer of geotextile fabric.

2) Sand Bed with Trench Design;
o Top layer of sand is to be 12-18 inches of 0.02 -0.04 inch diameter sand (smaller size is acceptable).
o Laterals to be placed in trenches with a covering of one-half to 2-inch gravel and geotextile fabric.
o The lateral pipes are to be underlain by a layer of drainage matting. 0 A presettling basin and/ or biofiltration swale is recommended to pretreat runoff discharging to the sand filter.
o A maximum spacing of 10 feet between lateral underdrain pipes is recommended.

Operation And Maintenance Recommendations

A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the sand filter to periodically clean the structure.
Scrape off sediment layer buildup during dry periods with steel rakes or other devices.
Replace some or all of the sand when permeability of the filter media is reduced to unacceptable levels that should be specified In the design of the facility.

Performance Standards

Sand Filtration Basins
o All runoff up to design volume is filtered through sand bed.
o The storage volume is based on runoff volume of the 1-inch rainfall event.
o Estimated long-term pollutant removal rates as follow:

Pollutant Removal Rate (%)

Primary Pollutants
Other Pollutants
Total Phosphorus

Lead
BOD
Sediment
Total Nitrogen
Zinc
COD
Bacteria
65
50-70
60
85
50
60-80
80
50-60

- Filtration System Performance Enhancement
o Sand/ peat beds have higher removal effectiveness due to adsorptive properties of peat.
o Designs incorporating vegetative cover on the filter bed increase nutrient removal.
- Pretreatment (sedimentation or oil and grease removal) will enhance the performance of the filter and will decrease the maintenance frequency required to maintain effective performance.

8.2.5 Constructed Wetlands

Standard Specifications For Constructed Wetlands

Required Specifications

- Inflow of water must be greater than that leaving the basin by infiltration or exfiltration.
- Designed for an extended detention time of 48 hours for the 1-inch rainfall event.
- The orifices used for extended detention will be vulnerable to blockage from plant material or other debris that will enter the basin with stormwater runoff. merefore, some form of protection against blockage shall be installed (such as some type of noncorrodible wire mesh).
- Surface area of the wetland shall account for a minimum of 3 percent of the area of the watershed draining into it.
- The length to width ratio shall be at least 2 to 1.
- A soil depth of at least 4 inches shall be used for shallow wetland basins.
- Approximately 75 percent of the wetland shall have water depths less than 12 inches, and 25 percent of the wetland shall have depths ranging from 2 to 3 feet.
- The deeper area of the wetland shall include the outlet structure so outflow from the basin is not interfered with by sediment buildup.
- A forebay shall be established at the pond inflow points to capture larger sediments and be 4 to 6 feet deep. Direct maintenance access to the forebay shall be provided with access 25 feet wide minimum and 5: l slope maximum. Sediment depth markers shall be provided.
- If high water velocity is a potential problem, some type of energy dissipation

device shall be installed.

- The designer shall maximize use of pre-and post-grading pondscaping design to create both horizontal and vertical diversity and habitat.
- A minimum of 2 aggressive wetland species (primary species -Figure 8-5) of vegetation shall be established in quantity on the wetland. (See Appendix C)
- Three additional wetland species (secondary species-Figure 8-5) of vegetation shall be planted on the wetland, although in far less numbers than the two primary species. (See Appendix C)
- 30 to 50 percent of the shallow (12 inches or less) area of the basin shall be planted with wetland vegetation.
- Approximately 50 individuals of each secondary species shall be planted per acre; set out in 10 clumps of approximately 5 individuals and planted within 6 feet of the edge of the pond in the shallow area leading up to the ponds edge; spaced as far apart as possible, but no need to segregate species to different areas of the wetland.
- Wetland mulch, if used, shall be spread over the high marsh area and adjacent wet zones (-6 to +6 inches of depth) to depths of 3 to 6 inches.
- A minimum 25-foot buffer, for all but pocket wetlands, shall be established and planted with riparian and upland vegetation (50 foot buffer if wildlife habitat value required in design). Surrounding slopes shall be stabilized by planting in order to trap sediments and some pollutants and prevent them from entering the wetland.
- A maintenance plan shall be provided and adequate provision made for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland shall be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/ or accumulation of plant material.

Recommended Specification

- It is recommended that the frequently flooded zone surrounding the wetland be located within approximately 10 to 20 feet from the edge of the permanent pool.
- Soil types conducive to wetland vegetation should be used during construction.
- The wetland should be designed to allow slow percolation of the runoff through the substrate (add a layer of clay for porous substrates).
- The depth of the forebay should be in excess of 3 feet and contain approximately 10 percent of the total volume of the normal pool.
- As much vegetation as possible and as much distance as possible should separate the basin inlet from the outlet.
- Of the 75 percent of the wetland that should be 12 inches deep or less, it is recommended that approximately 25 percent range from 6 inches deep to 12 inches deep, and that the remaining 50 percent be 6 inches or less in depth.
- The water should gradually get shallower about 10 feet from the edge of the pond.
- The planted areas should be made as square as possible within the overall design of the wetland, rather than long and narrow.
- The only site preparation that is necessary for the actual planting (besides flooding the basin) is to ensure that the substrate is soft enough to permit relatively easy insertion of the plants.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the wetland to periodically clean the structure. The maintenance agreement shall provide for ongoing inspection and maintenance, with more intense activity for the first three years after construction.
- The wetland shall be maintained to prevent loss of area of ponded water available for emergent vegetation due to sedimentation and/ or accumulation of plant material.
- Sediment forebays shall be cleaned every 2 to 5 years except for pocket wetlands without forebays that are cleaned after a six inch accumulation of sediment.
- The ponded water area may be maintained by raising the elevation of the water level in the permanent pond by raising the height of the orifice in the outlet structure, or by removing accumulated solids by excavation.
- Water levels may need to be supplemented or drained periodically until vegetation is fully established.
- It may be desirable to remove contaminated sediment bottoms or to harvest above ground biomas and remove it from the site in order to permanently remove pollutants from the wetland.

Performance Standards

- Performance depends on appropriate plantings for the soils, climate, and types of pollutants or land use (oil and grease, high sediment loads, high nutrient loads) in the drainage area.
- Design performance depends on protecting marsh-type plantings. Performance enhancement can be obtained by increasing the size of the marsh area, by incorporating multiple pools into marsh area, or by incorporating a network of shallow channels in the marshy area.
- Estimated long-term pollutant removal rates as follow:

- Primary
Pollutants
- Other
Pollutants
Pollutant
Total Phosphorus
Lead
BOD
Sediment
Total Nitrogen
Zinc
COD
Removal Rate (%)
50-60
75-85
50-60
90-99

40-50
75-85
55-65

8.2.6 Infiltration Trenches

Standard Specifications For Infiltration Trenches

Required Specifications

- Used in small drainage areas less than 15 acres.
- Drain the 1 inch rainfall storm in 48 hours.
- A minimum of one soils boring is required for every 50 feet of trench length, and no less than 2 soils logs for each proposed trench location. Borings should be taken to a depth of at least five feet below the trench depth.
- Each soils boring shall extend a minimum of 3 feet below the bottom of the trench, describe the NRCS series of the soil, the textural class of the soil horizon(s) through the depth of the log, and note any evidence of high ground water level, such as mottling. In addition, the location of impermeable soil layers or dissimilar soil layers should be determined.
- For runoff treatment, the soil infiltration rate should be between 0.5 and 2.4 inches per hour.
- Soil textures with minimum infiltration rates of 0.17 inches per hour or less are not suitable for infiltration trenches.
- Soils that have a 30 percent or greater clay content are not suitable for infiltration trenches.
- Soils that are suitable for infiltration systems are silt loam, loam, sandy loam, loamy sand, and sand.
- The use of infiltration systems on fill is not allowed due to the possibility of creating an unstable subgrade.
- A minimum of 3 feet difference is required between the bottom of the infiltration trench and the groundwater table and to bedrock.
- Site slope must be less than 20 percent, trench must be horizontal.
- The proximity of building foundations shall be at least 10 feet horizontally.
- A minimum of 100 feet from water supply wells shall be maintained when the runoff is from industrial or commercial areas.
- The design infiltration rate shall be equal to one-half the infiltration rate found from the soil textural analysis.
- Water quality infiltration trenches must be preceded by a pretreatment SMP.
- If a presettling basin precedes the trench, then the combination of both SMPs must be designed to drain the 1 inch rainfall design storm within 48 hours.
- The aggregate material for the trench shall consist of a clean aggregate with a maximum diameter of 3 inches and a minimum diameter of 1.5 inches.
- Stone aggregate backfill material for the trench shall have a maximum diameter of 3 inches and a minimum diameter of 1.5 inches. For design purposes, void space for these aggregates may be assumed to be in the range of 30 percent to 40 percent.
- The aggregate shall be completely surrounded with an engineering filter fabric. If the trench has an aggregate surface, filter fabric shall surround all aggregate fill material except for the top one foot.

- Runoff must infiltrate through at least 18 inches of soil, which has a minimum cation exchange capacity of 5 milliequivalents per 100 grams of dry soil.
- An observation well shall be installed for every 50 feet of trench length.
- The observation well shall consist of perforated PVC pipe, 4 to 6 inches in diameter, located in the center of the structure, and be constructed flush with the ground elevation of the trench.
- The top of the observation well shall be capped to discourage vandalism and tampering.
- Bypass larger flows

Recommended Specification

- Infiltration trenches work well for residential lots, commercial areas, parking lots, and open space areas.
- Can be installed under a swale to increase the storage of the infiltration system.
- Infiltration systems shall not be constructed until all construction areas draining to them are fully stabilized.
- An analysis shall be made to determine any possible adverse effects of seepage zones when there are nearby building foundations, basements, roads, parking lots, or sloping sites.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the infiltration trench to periodically clean the structure.
- The trench shall be monitored after every large storm (> 1 inch in 24 hours) for the first year after completion of construction and be monitored quarterly thereafter.
- Sediment buildup in the top foot of stone aggregate or the surface inlet shall be monitored on the same schedule as the observation well.

Performance Standards

- Full exfiltration trench
- Runoff can only exit the trench by exfiltrating through the stone into the underlying soils.
- The storage volume is based on runoff volume of the 1-inch rainfall storm.
- Estimated long-term pollutant removal rates as follows:

- Water quality trench
  - The storage volume is based on first flush volume of 1 inch of runoff from the contributing area.
  - Estimated long-term pollutant removal rates as follow:

- Primary Pollutants

- Other Pollutants

Pollutant	Removal Rate (%)
Total Phosphorus	60-70
Lead	85-90
BOD	80
Sediment	90
Total Nitrogen	55-60
Bacteria	90

- Partial exfiltration system
  - o The trench is not designed to rely completely on exfiltration to dispose of the captured runoff volume, a perforated pipe is used to dram part of the volume, being placed either beneath or near the top of the trench.
  - o The system is not as effective as a full exfiltration system.
  - o Addition of a layer of very sandy soil over the gravel trench results in removal rates as high as 60 percent for the suspended sediment and trace metal loads, 50 percent for oxygen demand, and 40 percent for nutrient loads.
- Pretreatment
  - o Pretreatment minimizes trench maintenance requirements.
  - o Suspended sediment loads, which will clog the trench, can be reduced by requiring that the stormwater runoff pass through a 20-foot grassed filter strip prior to entering the trench.
  - o Hydrocarbon loadings (oil and grease) that will clog the filter fabric and sand filter underlaying the trench can be reduced by the use of oil and grit chambers (when receiving large parking lot and roadway runoff).

8.2.7 Filter Strips And Flow Spreaders

Standard Specifications For Filter Strips And Flow Spreaders

Required Specifications

- The use of filter strips and flow spreaders shall be limited to drainage areas of 10 acres or less with the optimal size being less than 5 acres.
- Capacity of the spreader and/ or filter strip length (perpendicular to flow) shall be determined by estimating peak flow from the 25-year storm if the entire storm is routed through the spreader or the 1 inch first flush rainfall storm if this volume of flow is diverted to the spreader for water quality control.
- Drainage area into spreader shall be restricted so that maximum flow will not exceed 30 cfs.

- Primary Pollutants

- Other Pollutants

Pollutant	Removal Rate (%)
Total Phosphorus	65-75
Lead	95-99
BOD	90
Sediment	90-99
Total Nitrogen	60-70
Bacteria	98

- Channel grade for the last 20 feet of the dike or diversion entering the level spreader shall be less than or equal to 1% and designed to provide a smooth transition into spreader.
- Grade of level spreader shall be 0%.
- Depth of level spreader as measured from the lip shall be at least 6 inches.
- Appropriate length, width, and depth of flow spreader shall be selected from the following table.
- Level spreader lip shall be constructed on undisturbed soil (not fill material) to uniform height and zero grade over length of the spreader.
- Released runoff to outlet onto undisturbed stabilized areas in sheet flow and not allowed to reconcentrate below the structure.
- Slope of filter strip from level spreader shall not exceed 10 percent.
- All disturbed areas shall be vegetated immediately after construction.
- Filter strip width to be a minimum of 20 feet.

Recommended Specifications

- Top edge of filter strip shall directly abut the contributing impervious area and follow the same elevational contour line.
- Runoff water containing high sediment loads to be treated in a sediment trapping device before release in a flow spreader.
- Spreader lip to be protected with erosion resistant material, such as fiberglass matting or a rigid non-erodible material for higher flows, to prevent erosion and allow vegetation to be established.
- Wooded filter strips are preferred to gravel strips.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the filter strip/ flow spreader to periodically clean the structure.
- Flow spreader shall be inspected after every rainfall until vegetation is established, and needed repairs made promptly.
- After area is stabilized, inspections shall be made quarterly.
- Vegetation shall be kept in a healthy, vigorous condition.
- Filter strip and flow spreader shall be maintained in a manner to achieve sheet flow.

Performance Standards

- General Performance Information
o Filter strips must accept stormwater runoff as overland sheet flow in order to effectively filter suspended materials out of the overland flow.
o In order to function properly, the strip should be at least as wide as the flow path entering the filter, and flow entering a filter strip must be spread relatively uniformly over the width of the strip.
o The removal of soluble pollutants is low because the degree of infiltration provided is generally very small.
o Removals of nutrients and oxygen demand decrease as the amount of clay in the soil increases.
o Filter strip applications should be limited to drainage areas of 10 acres or less, with the optimal size being less than 5 acres.
o The use of filter strips to treat parking lot runoff or street runoff should incorporate a level spreading device such as a shallow stone filled trench or slotted parking blocks.

- 20-Foot Wide Grassed Filter Strip
o Minimal pollutant removal. This design primarily removes the coarser suspended particles in runoff by the lowering of runoff velocities.
o Pollutant removal enhanced by mild slopes, minimal mowing/ maintenance, sustaining natural cover if possible.
o Long term estimated removal of pollutants is as follows:

- Primary
Pollutants

- Other Pollutants

Pollutant
Total Phosphorus
Lead
BOD
Sediment
Total Nitrogen
COD

Copper
Zinc
Removal Rate (%)
10
30
10
30
10
10
30
30

- 100-Foot Wide Grassed Filter Strip
  - o Maximal natural pollutant removal. This design removes both fine and coarse suspended particles in runoff by lowering runoff velocities over a significant length of flow path.
  - o Pollutant removals can be enhanced by mild slopes, minimal mowing/ maintenance, sustaining natural cover if possible.
  - o Long term estimated removal of pollutants is as follows:

- Primary
Pollutants
- Other Pollutants
Pollutant
Total Phosphorus
Lead
BOD
Sediment
Total Nitrogen
COD
Copper
Zinc
Removal Rate (%)
50
90
70
90
50
70
90
90

8.2.8 Grassed Swales

Standard Specifications For Grassed Swales

Grassed swales are also described as biofiltration swales with the major difference being that grassed swales often have check dams where biofiltration swales do not.

Required Specifications

- Grassed swale shall only convey standing or flowing water following a storm.

- As a water quality SMP, grass swales shall be designed for the 1 inch rainfall design storm. If the entire channel design storm is to be accommodated in the swale (e. g., 25-year) then the swale shall be designed for this event.
- Limited to peak discharges generally less than 5 to 10 cfs.
- Limited to runoff velocities less than 2.5 ft/ s.
- Maximum design flow depth to be 1 foot.
- Swale slopes shall be graded as close to zero as drainage will permit.
- Swale slope shall not exceed 2%.
- Swale cross-section shall have side slopes of 3: 1 (h: v) or flatter.
- Underlying soils shall have a high permeability (fc > 0.5 inches per hour).
- Swale area shall be tilled before grass cover is established.
- Dense cover of a water tolerant, erosion resistant grass shall be established over swale area.
- To obtain credit as a water quality SMP, grassed swales must have a minimum length of 100 feet.

Recommended Specifications

- As a SMP, grassed swales are limited to residential or institutional areas where percentage of impervious area is relatively small.
- Seasonally high water table to be greater than 3 feet below the bottom of the swale.
- Check dams can be installed in swales to promote additional infiltration. Recommended method is to sink a railroad tie halfway into the swale. Riprap stone should be placed on the downstream side to prevent erosion.
- Maximum ponding time behind check dam to be less than 48 hours.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the grassed swale to periodically clean the structure.
- Grass swale shall be maintained to keep grass cover dense and vigorous.
- Maintenance shall include periodic mowing, occasional spot reseeding, and weed control.
- Swale grasses shall never be mowed close to the ground.
- Fertilization of grass swale shall be done when needed to maintain the health of the grass, with care not to over-apply the fertilizer.

Performance Standards

- General Performance Information

o Grassed swales provide a water quality benefit by filtering suspended material out of the overland flow. They have little or no value at removing soluble pollutants because the degree of infiltration provided is generally small.

o In order to function optimally, a grassed swale must be in an area where its longitudinal slope is very slight (2% or less). The table below shows the low removal rates for grassed swales on a 5 percent slope. If discharges or velocities are greater than those recommended (greater than

10 cfs or 2.5 ft/ s, respectively), the ability of the swale to perform as a water quality SMP is severely impaired.
o The use of check dams in the swale helps to lower the discharge velocity and can, in some cases, allow their beneficial use in situations where the swale slope is greater than recommended. l Rainfall events of less than 0.25 inches may show increased removals due to the slower velocities in the swales.

Grassed Swales on a 5% Slope
o Long term estimated removal of pollutants is as follows: Pollutant
Primary
Pollutants
Other Pollutants
Pollutant
Total Phosphorus
Lead
BOD
Sediment
Total Nitrogen
COD
Copper
Zinc
Removal Rate (%)
10
10
10
10
10
10
10
10

Grassed Swales on a Slope Less Than 5% With Check Dams
o Long term estimated removal of pollutants is as follows:

Primary
Pollutants

Other Pollutants

Pollutant
Total Phosphorus
Lead
BOD
Sediment
Total Nitrogen
COD
Copper

Zinc
Removal Rate (%)
30
10
30
30
30
30
10
10

8.2.9 Oil/ Grit Separators

Standard Specifications For Oil/ Grit Separators

Required Specifications

- Separators shall be sized for the 1 inch rainfall design storm. Larger storms shall not be allowed to enter the separator.
- Separator shall be structurally sound and designed for acceptable traffic loadings where subject to traffic loadings.
- Separator shall be designed to be watertight.
- Volume of separator shall be at least 400 cubic feet per acre tributary to the facility (first two chambers).
- Forebay or first chamber shall be designed to collect floatables and larger settleable solids. Its surface area shall not be less than 20 square feet per 10,000 square feet of drainage area.
- Oil absorbent pads, oil skiers, or other approved methods for removing accumulated oil shall be provided.
- Separator pool shall be at least 4 feet deep.
- Weirs, openings, and pipes shall be sized to pass as a minimum a 2-year storm.
- Manholes shall be provided to each chamber to provide access for cleaning.

Recommended Specifications

- Oil absorbent pads, oil skimmers, or other approved methods for removing accumulated oil shall be provided.
- Separator to be located close to the source before pollutants are conveyed to storm sewers or other SMPs.
- Use only on sites of less than one acre.
- Provide perforated covers as trash racks on orifices leading from first to second chamber.
- Use three chambers for treatment similar to Figure 8-12.
- Center chamber may contain a coalescing medium to enhance the gravity separating process.
- Storm drain inlet in third chamber to be located above floor to permit additional settling.
- Stormwater from rooftops and other impervious areas not likely to be polluted with oil shall not discharge to the separator.
- Design to bypass flows above 400 cubic feet per acre.

Operation And Maintenance Recommendations

- A stormwater management easement and maintenance agreement shall be required for each facility. The maintenance covenant shall require the owner of the separator to periodically clean the structure.
- Cleaning quarterly shall be a minimum schedule with more intense land uses such as gas stations requiring cleaning as often as monthly.
- Cleaning shall include pumping out wastewater and grit and having the water processed to remove oils and metals.

Performance Standards

- Oil & Grease Separator Performance
  - o These devices are for the most part ineffective as a stand alone treatment of stormwater runoff quality, unless hydrocarbons are the only pollutant of concern.
  - o Hydrocarbons in urban runoff can effectively clog the infiltration capacity of underlying soils because they tend to attach themselves to particles in the water column and settle to the bottom of the SMP.
  - o The removal of hydrocarbons will extend the maintenance interval required for downstream SMPs by removing these substances that impair their effectiveness.
- Primary Pollutants -Phosphorus, Lead, BOD

  - o Performance standards as related to phosphorus, lead, and BOD are not relevant.

8.3 Stream bank Restoration

Although effective watershed runoff controls are needed to eliminate the root causes of stream degradation, stream health can also benefit from restoration efforts that directly target the stream channel and stream banks. Stream bank erosion needs to be halted, and both in-stream and riparian habitats restored. Such a program requires expertise in areas such as stream forming processes, slope stabilization, plant science,, and aquatic biology. Local Soil Conservation Service and Fish and Wildlife Service staff may be able to, provide some of this expertise. Stream restoration includes three major activities:

- Riparian reforestation
- Streambank stabilization
- Streambed restoration

Riparian Reforestation

The contribution of trees and woody under story vegetation to the maintenance of stream health cannot be overstated. Streamside forested areas not only provide habitat, shade, and forage for both aquatic and land-based species, but their ability to filter pollutants and rainfall provides a buffer, a last line of defense, from watershed runoff. A program to restore forested streamside areas should receive early consideration, because it can be one of the most cost-effective steps that a community takes in its stream restoration efforts. The objective should be to replicate or mimic the natural ecosystem as much as possible, so mixed-age native plant and tree species are preferred. Encouraging participation by citizen volunteers can increase the cost effectiveness of the program. Though most revegetation efforts focus on streambanks, the hydrologic characteristics of the watershed can be improved by upland reforestation as well.

Streambank Stabilization

Anyone faced with an eroding or collapsing streambank needs first to determine the cause of the problem. Streambank erosion occurs for a number of reasons, including increased stream velocity, obstacles in the stream, floating debris, wave action, and direct rainfall. Streambank failure occurs when a large section of streambank collapses into the stream channel. Among the causes of streambank failure are changes in channel cross-section through down-cutting of the streambed and undercutting of the bank, increased load on the top of the bank, and internal pressure from uneven water absorption.

Selection of an appropriate bank stabilization method requires careful analysis of each site. No single method requires careful analysis of each site. No single method is appropriate in all situations. Technical advice will often be needed, and is available from sources such as the local Soil Conservation Service and Cooperative Extension Service offices, or from private consultants. One important note: a Corps of Engineers permit may be needed before any material is placed in a stream or adjacent wetlands. The Corps of Engineers Savannah District office should be contacted (l-800448-2402 or 1-912-652-5347.)

Detailed discussion of the many possible stream and streambank stabilization techniques is beyond the scope of this Manual, but one general approach needs to be mentioned because of growing realization of its contribution to the overall health of streams. The approach has been called to bioengineering or biotechnical approach. Its aim is to replicate or reintroduce natural stream and slope stabilization processes as much as possible. The biotechnical approach to slope protection combines the use of mechanical (or structural) elements with biological elements (plants), functioning together and mutually reinforcing each other (Gray and Leiser, 1982). Biotechnical techniques in which plant materials are the primary structural component have come to be identified by the term .soil bioengineering.. Techniques include installing plantings of woody vegetation such as willows, either as individual live cuttings, or in bundles of cuttings. If planted correctly and given time to establish root systems, the cuttings can grow into a dense network of protective vegetation that can bend but not break under stress and that is self-repairing. The vegetation.s root matrix provides resistance to the sliding and shear displacement forces involved in slope erosion.

Although .living construction. methods have been systematically studied and used in Europe for more than half a century, technical information on such methods became easily available in the United States only recently. The Pennsylvania Scenic Rivers Program.s 1986 guidebook for landowners is an attractive, reader-friendly publication (Pennsylvania Scenic Rivers Program, 1986). The Izaak Walton League of America has published a 21-page survey of stream-bank stabilization methods (Izaak Walton League of America, 1989). The Washington Department of Ecology.s draft Stormwater Management Manual for the Puget Sound Basin includes bioengineering methods among the many groups of protection measures that it describes (Washington Department of Ecology, 1992). Also, the Georgia Soil and Water Conservation Commission plans to include a detailed description of the concepts in a new guidebook titled Controlling Streambank Erosion which will soon be available.

Gray and Leiser list four reasons to prefer biotechnical approaches:

- Their cost effectiveness. Lower cost vegetative treatments can reduce the amount of higher cost structural treatments that may be needed.

- Their environmental compatibilitv. Biotechnical systems tend to blend into the landscape and are less visually intrusive. Examples include log or timber cribs gabion and rock breast walls, and reinforced earth. In addition, wherever possible, vegetation is incorporated into the structures, for example by planting in the spaces between structural members.

- Their use of indigenous, natural materials. Wherever possible, natural locally available materials are used: earth, rock, timber, vegetation -in contrast to man-made materials such as steel and concrete.
- Their labor-and skill-intensiveness. Well-supervised, skilled labor can often be substituted for high-cost, energy-intensive materials (Gray and Leiser, 1982).

Most importantly, biotechnical methods contribute to the support and protection of the ecology of a stream in ways that purely structural techniques do not.

If possible, a qualified bioengineer should be consulted to evaluate site conditions and determine the appropriated mix of measures that will adequately solve the problem and stand up to the test of time. In some cases, a solely vegetative approach may be all that is needed. In others, conditions such as excessive stream velocities or poor soil conditions may require a combination of vegetative and structural elements. And in still others, space limitations or other conditions may require a solely structural approach such as stonewalls or bulkheads. Some of the most common conditions that may preclude the soil bioengineering preference for vegetative measures include inadequate space, heavy pedestrian traffic, and the need for an unobstructed view. and too much shade.

Streambed Restoration

Prior to any streambed restoration, upstream conditions should be assessed. Without corrective measures or retrofitting upstream, stormwater flows could quickly destroy any restoration work. If the steam is in equilibrium, or if appropriate corrective measures are in place, streambed restoration can recreate the habitat conditions needed to support aquatic life. Several factors may need to be addressed in streambed restoration:

- Replacement of pools and riffles.
- Velocity control.
- Restoration of the stream gradient and normal flow channel.
- Removal of major stream obstructions.
- Restoration of suitable channel patterns. There are three major channel patterns: (1) meandering -which is characterized by repetitive bends, (2) irregular -which is more or less straight; and (3) braided -which separates and rejoins around islands. Which pattern is appropriate depends upon surrounding soil and slope conditions, as well as the original stream patterns (Dunne, 1978).
- Restoration of the substrate (removal of sediment and replacement with gravel and cobbles, as appropriate for the streams).
- Restoration of adjacent wetlands and floodplains.

The number of factors affected by restoration and the extent of the measures taken will depend upon individual stream conditions. Some techniques permit the stream flows themselves to work to restore healthier streambed conditions; others require excavation and physical realignment of the stream channel. Three basic techniques include deflectors, in-stream boulders and drop structures. With many variations, these techniques are used throughout the country.

Reflectors can be easily constructed of common, local material such as cobbles, boulders and logs, and are adaptable to a variety of conditions and stream sizes. They are sited in the channel with the intent of deflecting the current into a narrower channel. Deflectors can use the stream flow for a variety of purposes, including deepening channels, developing downstream pools, enhancing pool riffle ratios and assisting in the restoration of meander patterns with channelized reaches. There are several deflector designs, such as simple double .wing deflector. that consists of rock structures on each bank deflecting the streamflow to a central channel, single deflectors along one bank, deflectors offset on opposite banks of a stream to imitate meanders, and V-type deflector, which is placed in the middle of the channels with the point of the .V. pointing upstream deflecting water towards both banks. This type of deflector helps reestablish riffles and pools downstream. An underpass deflector is a log placed across a small stream several inches off the bottom. Water is deflected under the log that helps remove sediment deposits and restore pools. (Gore, ed., 1985) (Rumble, 1990).

Drop Structures include a number of variations such as weirs, check dams, sills and plunges. They can serve a variety of functions in streambed restoration depending upon their design, including: slowing streamflow; deepening existing pools; and creating new pools upstream and downstream. Structures with notches can be used to control heavy stormwater flows and can help re-establish deep pools immediately downstream. Drop structures can be made of concrete, logs or boulders. Log or boulders structures can be used to replicate small falls or rapids. Single log dams across a stream bed are simple and effective in restoring plunge pools. The K-dam is a variant of the single log dam, so named, by added downstream bracing. In some areas, especially headwater areas, reintroducing beavers has been effective in restoring habitat (Gore, ed., 1985). Their dams functions as drop structures in headwaters and on small streams.

Boulder placement is a third in-channel treatment that can assist streambed restoration. Boulders can be used to reduce velocity, restore pools and riffles, restore meanders, provide cover and protect eroded banks by deflecting flow (Gore, ed., 1985). Boulders can be placed randomly or in a pattern. Placing them in a .V. pointed upstream produces eddies that replicate riffles as well as restores downstream pools. Combined with placement of cobbles and gravels, boulder placement can also help restore the stream substrate.

Excavation and fill may also be necessary to restore the stream gradient, the normal flow channel and the stream channel pattern, including meanders and braids, where appropriate. Channel pattern restoration should be combined with streambank restoration and re-vegetation. Streams that have been severely degraded by large amounts of sediment or heavy stormwater flows may require greater restoration work. Sediment may have to be removed mechanically and replaced with gravel and cobbles to replicate the original streambed. Major debris accumulation that is obstructing flows may also need removal.

Restoration of riparian wetlands and floodplains can also be included in re-vegetation project with special consideration given to planting species appropriate to the specific site conditions, including soil types, and degree of saturation. The following can provide further information on streambed restoration:

- Guidelines for Streambank Restoration. State Soil & Water Conservation Commission, 1994.

- Soil Conservation Service Engineering Field Book, Part 650, 1992.

- The Restoration of Rivers and Streams. James A. Gore, Editor, 1985.

- Stream Restoration Along the Greenways in Boulder, Colorado. John L. Bamett, 1991.

- The State of the Anacostia 1989 Status Report, Peter A. Kumble, 1990.

- A Streambank Stabilization and Management Guide for Pennsylvania Landowners. Commonwealth of Pennsylvania, Department of Environmental Resources, 1986.

- Stream Obstruction Removal Guidelines. Wildlife Society and American Fisheries Society, 1983

8.4 References

American Association Of State Highway And Transportation Officials, Model Drainage Manual, 1992.

Georgia Soil and Water Conservation Commission, Manual For Erosion And Sediment Control In Georgia, Fourth Edition, P. O. Box 8024, Athens, Georgia 30603, 1996.

Maestri, B. and Others, Managing Pollution From Highway Stormwater Runoff, Transportation Research Board, National Academy of Science, Transportation Research Record Number 1166, 1988.

Metropolitan Washington Council of Governments, A Current Assessment Of Urban Best Management Practices -Techniques for Reducing Non-Point Source Pollution in the Coastal Zone, 777 North Capital Street, Suite 300, Washington, D. C., 1992.

State of North Carolina, Erosion And Sediment Control Planning And Design Manual, North Carolina Sedimentation Control Commission, North Carolina Department of Natural Resources And Community Development, 1988.

8.5 Appendix A -Example Design Applications

A.1 Example Design -Infiltration Trenches

All equations used in the following section are fully documented and discussed in Maryland.s Department of Natural Resources Standards and Specifications for Infiltration Practices, 1984 (See Reference #2).

Site Layout

The site for an infiltration trench consists of two areas:

1. the portion of the watershed that contributes direct runoff to the infiltration trench, which is denoted as Au and
2. the portion of the watershed allocated to the basin (does not contribute runoff to the trench), which is denoted as Ab. The subscript u and b are used to indicate the upland and basin drainage areas, respectively.

Design Procedure

Step 1 Calculate the volume of runoff, in inches, from the first l-inch of rainfall over the total impervious surfaces included within the proposed development, .Qu.

Step 2 Compute the maximum allowable trench depth (dmax from the feasibility equation 8. 1.

Step 3 Select the trench design depth (dt) based on the depth that is at least two feet above the seasonal high groundwater table, or a depth less than or equal to &, whichever results in the smaller depth.

Dmax=Fts/Vr (8.1)

Where: F = minimum infiltration rate, in/ hr
ts= storage time, hr
Vr = void ratio in soil or rock

Step 4 Compute the trench surface area (At) from Equation 8. 2:

At= (.QuAu)/(Vrdt . P + Fts) (8.2)

Where: At = surface area of trench, ft2

P = rainfall depth, ft
Other variables previously defined

Step 5 Compute the trench width or length equation 8.3.

Lt = (.QuAu)/(Vrdt -P + Fts)Wt (8.3)

Where: Lt = length of trench, ft
Wt = width of trench, ft
Other variables previously defined

In the event that the side walls of the trench must be sloped for stability during construction, the surface dimensions of the trench area should be based on equation 8.4:

At= (Lt . Zdt)(Wt . Zdt) (8.4)

Where: Z = trench side slope ratio

The design procedure would begin by selecting a top width (Wt) that is greater than 2Zdt, for a specified side ratio (Z). The length (Lt) is then determined as:

Lt = Zdt + (At)/(Wt . Zdt) (8.5)

Example Application An infiltration trench with surface inlets will be used to control the first inch of rainfall from the impervious surfaces within a 6-acre commercial site. Calculations for the surface area of the trench. Following are the

Design Data:
F = 1.02 in/ hr
Vr = 0.4
ts = 48 hours
Depth to groundwater = 12 feet
Depth to bedrock = 18 feet

Step 1 Calculate the volume of runoff, in inches, from the first l-inch of rainfall over the total impervious surfaces included within the proposed development, .Qu.

For the proposed development, 63 percent (0.63) of the total development is covered by impervious surfaces. Thus the volume that must be controlled would be as follows:

.Qu = l-inch x 0.63 = 0.63 in.

Step 2 Compute the maximum allowable trench depth (dmax)) by the feasibility formula:

dmax, = Fts/Vr

dmax = (1.02)(48)/ 0.4 = 122.4 in = 10.2 ft

For this example the depth to the groundwater table is 12 feet and the depth to bedrock is 18 feet.

Step 3 Select a trench design depth less than dmax, and at least two feet above the groundwater table (subtract 1.5 feet for the overlying soil cover with surface inlets).

Select dt = 3.0 ft

Step 4 Compute the trench surface area (At) by the equation:

At = (.QuAu)/(Vrdt . P + Fts)

Where: At = 6 acres x 43560 ft2/ acre = 261,360 ft2
Vr = 0.40
ts= 48 hours
F = 1.02 in/hr
dt = 8.5 ft
.Qu= 0.63 in = 0.053 ft
P= 1 in. =. 08ft
At = [(0.053)(261,360)]/[(0.40)(3) -.08 + (l. 02/ 12 x 48)]
At = 2,664 ft2

A. 2 Example Design -Grassed Swales

Site Layout

The site layout will consist of the portion of the watershed that contributes direct runoff to the swale area or the upland area, which is denoted as Au, and the portion of the watershed allocated for swale storage, which is denoted as As. It is important to note that the upland area (Au) does not include the area allotted to the swale surface (As). Swale locations are usually either on the side or back of the property line or along the side of roadways (not in the road right-of-way). Installation of berms or check dams at certain intervals along the length of the swale will result in storage.

Design Procedure

Step 1 Calculate the volume of runoff, in inches, from the first l-inch of rainfall over the total impervious surfaces included within the proposed development, .Qu.

Step 2 Compute the maximum allowable swale check dam depth (dmax from the feasibility equation 8.8.6.

Step 3 Select the swale design depth (ds) based on the depth that is at least two feet above the seasonal high groundwater table, or a depth less than or equal to dmax, whichever results in the smaller depth.

dmax = Fts/Vr = FTp (8.6)

Where: F = minimum infiltration rate, in/ hr
ts = maximum storage time for stone aggregate reservoir, hr
Vr = void ratio in soil or rock
Tp = maximum allowable ponding time for surface storage, hr

Step 4 The swale surface area dimensions can be determined from Equations 8. 8. 7 and 8. 8. 8. The bottom width (Wb) is selected along with the side slope ratio (Z), and depth of check dam (ds). The swale top width (W) and total hydraulic length (Lr) may be computed as:

W = Wb + 2dsZ (8.7)
LT = [.QuAu]/[(ds/4)(W + Wb) = W(Fts . P)] (8.8)

Where: P = rainfall depth
Other variables defined above.

Step 5 The maximum required spacing between check dams is computed as:

L= ds/Ss, (8.9)

Where: S, = bottom slope of swale, ft/ ft

Step 6 The number of check dams needed to impound and store the runoff volume is determined as:

Ns = LT/L

Where: L = length of swale behind each check dam, ft

Step 7 If LT, is restricted by the site layout, the level of control provided by the swales is determined by:

QS= VW /Au (8.11)

Where: VW, = [ds (W + Wb)L][Ns]/4
Qs = runoff storage depth
VW = volume of swale storage

Example Application

Low density residential lots of 314 acres are to be developed. A total of 4 lots will be created. The site will be designed to be managed with grassed swales with check dams located along the back of the lots. The total area of the development is 3.0 acres with 20% impervious area and 80% pervious area. Following are the calculations for the swale design.

Design Data: F = 2.41 in/ hr
Vr = 0.4
TP = 48 hours
Depth to groundwater = 12 feet
Depth to bedrock = 14 feet

Step 1 Calculate the volume of runoff, in inches, from the first l-inch of rainfall over the total impervious surfaces included within the proposed development, .Qu.

For the proposed development, 48 percent (0.48) of the total development is covered by impervious surfaces. Thus the volume that must be controlled would be as follows:

.Qu= l-inch 0.48 in.

Step 2 Compute the maximum allowable swale depth (dmax) from the feasibility equation:

dmax = 2.41(48) = 115.7 in. = 9.64 ft

Step 3 Select the swale check dam design depth (ds); the depth to the groundwater table and bedrock is greater than dmax.

ds = l. 0 ft

Step 4 Select the swale bottom width (Wb) and the swale side slope ratio (Z), assuming the same side slopes. Determine the top width (W) of the swale check dam from:

W= Wb + 2dsZ

Where: Z = 5
Z= 5h/ l v
Wb= 18ft
W = 18 + 2(1.0)(5) = 28 ft

The site layout allows for a swale length of 480 feet along the back of all the lots. The total swale length (LT) is fixed so that the volume of swale storage (Vw) may be determined from the dimensions given:

VW = [ds(W + Wb)LT]/4

VW = [1.0(28+18)480]/4 = 5,520 ft3

Step 5 The number of swale check dams (Ns) that needs to be constructed to achieve the volume of storage over the total swale length will vary with the depth of each check dam (ds), given as:

Ns = LT-/L

where L is the length of swale behind each check dam, given as:

L = ds/Ss,

Note: Ss = bottom slope of swale = 0.02 ft/ ft

L = 1.0/ 0.02 = 50.0 ft

Step 6 Ns = 480/ 83.3 = 9.6 (use 10 with an adjusted L of 48.0 feet*)

* The adjusted length is determined as L = LT/Ns

Thus, the design storage is 5,520 ft3, which is greater than the required storage of 5,227 ft3 (0.48 in x l ft/ l2in x 3 acres x 43560 ft2/ acre) so that sufficient storage is provided by the design.

For further information on the design of infiltration measures the following references are recommended.

Stormwater Infiltration Structure Design, National Stone Association, 1415 Elliot Place, NW, Washington, D. C., 1994. Available from McTrans, University of Florida, Gainesville, Florida, 32611, (352) 392-0378. Includes a computer model for infiltration design.

Maryland Department of Natural Resources, Water Resources Administration, Stormwater Management Division, Standards and Specifications for Infiltration Practices, 1984.

8.6 Appendix B -Guide for Maintenance of SMP Facilities

Introduction

Although the actual time that a Stormwater Management Practice (SMP) facility performs its design function is relatively brief (during and immediately following a storm event), it must constantly be ready to do so. This is due to the random nature of rainfall events and the impracticality of inspecting the facility and performing maintenance immediately prior to them. Additionally, pollutant removal efficiencies will decline over time if adequate maintenance is not performed. To maintain maximum pollutant removal, it is important to have SMPs fully operational at all times. To provide this operational level, the SMP facility operator must establish and sustain a comprehensive, regularly scheduled maintenance program.

The positive aspect of a properly functioning SMP facility is that it enhances downstream environments by mitigating the environmental impacts of land development; conversely, SMPs can diminish the positive impact on the environment if they are not properly maintained.

The following criteria provide a guide for maintenance of a SMP facility. These include access and maintenance easements, routine inspection of outlet structures, sediment disposal, maintenance agreements, and other considerations specific to SMP.s.

1. Access for Maintenance

Access for inspections, maintenance personnel, and equipment must be provided to all areas of a SMP facility that require observation or maintenance. The location and configuration of easements must be established during the construction of the facility, and maintained on a regular basis. The areas requiring access include the dam embankment, emergency spillway, side slopes, inlets, sediment forebays, riser structures, SMP devices, and pond outlets. In order to provide access for heavy equipment, a suitable 20 ft. wide roadway within a 25 ft. wide cleared access easement must be provided to the SMP facility. A typical roadway would consist of # 6 of stone compacted
subgrade (95% Maximum Theoretical Density). On large SMP facilities, additional easements to both upstream and downstream areas should be provided for maintenance access and additional improvements such as all-weather roads, access restrictions, and vandalism deterrents should be considered.

2. Sediment Accumulation and Removal

Sediment accumulation resulting from the normal operation of SMP measures must be recognized. Accommodations should be made for the removal and disposal of sediments. Disposal should be provided either onsite in reserved areas, used as fill or topsoil supplement, or removed from site.
For larger SMPs, access must be provided for equipment to dredge or otherwise remove accumulated silt materials since offsite disposal would likely be more necessary.

3. Maintenance Agreements

An agreement stating maintenance responsibility, schedule, and operations must be included in the approved plan submission. Easements for non-City maintained SMPs should include provisions to permit City inspection and maintenance (including reimbursement to the City agency for incurred costs) if a Property Owner or Homeowner Association or Homeowner Associations fails in its inspection and maintenance responsibility and creates a public nuisance. A maintenance agreement is attached.

4. Operation and Maintenance Costs

It is clear that the maintenance needs of SMPs are somewhat site specific, and the costs of conducting needed maintenance will vary accordingly. However, it is possible to determine cost estimates using some general SMP facility maintenance parameters. The operation and maintenance of a SMP facility will usually involve routine and non-routing maintenance procedures. Routine maintenance procedures will include inspections, debris and litter control, mechanical components maintenance, vegetation management, and other routine tasks as determined for the specific facility. Non-routine costs are those associated with removing accumulated sediments from the facility and long term structural repairs. Non-routine maintenance costs will vary greatly depending upon the size and depth of the facility, the volume of sediment trapped in the SMP facility, the accessibility of the SMP facility, and whether or not onsite disposal of the dredged sediments is possible.

The average annual operation and maintenance costs of an extended detention dry pond are estimated at 3-52 of the capital cost of the facility. The average annual operation and maintenance costs of a wet pond are estimated at 3 % of capital costs. Probable costs for a wet pond less than 100,000cubic feet is 5% of capital costs, while the probable costs for a wet pond greater than 1,000,000 cubic feet is 1% of capital costs. Initial capital costs will vary considerably depending on the type and size of the SMP facility.

While these cost estimates provide a general guideline to annual operation and maintenance costs, the Property Owner or Homeowner Association(s) of the SMP facility should plan ahead to ensure that funds are available when non-routine expenses are necessary. The costs of maintaining a SMP facility over the long run can be considerable, particularly when dredging or performing other non-routine maintenance. To lessen the immediate financial impact of these non-routine costs, it is strongly advised that any party responsible for SMP facility maintenance create a sinking fund for this eventuality. For extended detention dry ponds, which need to have sediment removed every 2 to 10 years, 10% to 50% of the anticipated dredging costs should be collected per year. For wet ponds, which need to be dredged every 5 to 15 years, approximately 6% to 20% of the anticipated costs should be accrued per year. Present value of the assessment can include anticipated interest.

5. Maintenance Specific to SMPs

SMP.s experience conditions which can lead to degraded efficiency and objectionable conditions. Areas of concern include: excessive weed growth, maintaining adequate vegetative cover, sedimentation, bank erosion, insect control, outlet stoppages, soggy surfaces, algal growth, fence maintenance, unsatisfactory emergency spillway, and dam failures/ leakages. The main problem for extended detention dry ponds is a tendency for a soggy bottom, which hinders facility maintenance and the growth of effective vegetative cover. (A) Inspections Scheduled, periodic inspections should provide the foundation for a comprehensive maintenance program. Detailed inspections, occurring at least annually, should be conducted by a qualified inspector to ensure that the facility is operating as designed and to provide a chance to schedule any maintenance which the facility may require. The American Public Works Association recommends that the following items be checked as minimum inspection requirements.

RECOMMENDED MINIMUM INSPECTION REQUIREMENTS

- Dam settling, woody growth, and signs of piping.

- Signs of seepage on the downstream face of the embankment.

- Condition of grass cover on the embankment, pond floor, and perimeter.

- Riprap displacement or failure.

- Outlet controls, debris racks, and mechanical and electrical equipment.

- Outlet channel conditions.

- Safety features of the SMP facility.
- Access for maintenance equipment.

If possible, inspections should be made during periods of wet weather to ensure that the SMP facility is maintaining desirable retention times. An inspection checklist is attached.
In addition to regularly scheduled inspections, the opportunity should be taken to note deficiencies during any visits by maintenance personnel. After major storm events the facility should be checked for clogging of the outlet structure.

(B) Sediment Accumulation

Typically SMP.s are designed to provide effective pollutant removal capabilities by enhancing sediment deposition. Periodic sediment removal is important to the effectiveness of these facilities; therefore, a schedule of sediment removal should be established.

(C) Vegetative Cover

If allowed to become established, small trees and brush with woody root systems can grow to cause destabilization and seepage in pond embankments which may result in the structural failure of the SMP facility. For this reason the dam embankment, side-slopes, and emergency spillway of a SMP facility should be kept free of woody growth and undesirable vegetation. This will require periodic mowing and a policy of not allowing plantings on these facilities. The frequency of mowing may need to be greater if the SMP facility is located in an area of high visibility. However, if possible, the SMP facility should be managed as an upland meadow with grass no shorter than 6-8 inches. Keeping grass much shorter than this can cause areas of the turf to die off or require a much higher level of maintenance.

Gradual slopes are necessary for establishing vegetative cover and for ease of mowing. Guidelines recommend a maximum of 3h: lv slopes for areas to be maintained by mowing.

The pool and bank slopes should be shallow enough to allow for dredging and mowing equipment and the pond bottom should have sufficient slope (maximum 2%) to avoid areas of ponded water. In poorly drained soils, low-flow concrete trenches will be required to help prevent long-term saturated conditions.

Erosion and bare areas noted during site visits should be backfilled with topsoil, compacted, and re-seeded. These problems, if taken care of promptly, can help to avoid more costly repairs made necessary by continued erosion of unstabilized soils.

No trees, brush, or other woody vegetation should be allowed to grow within 10 feet of the embankment or side slopes. Any old growth, and its root system, should be completely removed. The excavation should then be filled, compacted, re-seeded, and protected until properly vegetated. Any seedlings or planting should be removed-at the earliest opportunity and the disturbed areas properly stabilized.

(D) Shorelines

To minimize the maintenance of the area surrounding the shoreline of the SMP facility, the slopes should be relatively flat and bank stabilization materials such as riprap and vegetative growth cover should be incorporated into the design. As a minimum requirement, areas of the shoreline which are adjacent to the embankment of those areas that are most subject to wind erosion must be properly stabilized. The layer of stones should be 12 inches thick and placed on a 10 inch bed of gravel and should extend 3 feet below the normal pool elevation. It may also be necessary to add a slight berm on the upstream face of the dam to support the riprap and prevent it from slipping.

(E) Structural Repairs

The inlet, outlet, and riser structures of the SMP facility should preferably be constructed of precast or reinforced concrete because of its greatly extended service life. Perhaps the largest single expense involved in SMP facility maintenance will be the eventual repair or replacement of these parts of the SMP facility. The use of quality materials with a long service life should be used.

6. SMP Facility Maintenance ResponsibilityGuidelines

It is the policy of the City of Loganville that all stormwater quality management SMPs and all stormwater detention facilities be maintained by the Property Owner or Homeowner Association. Maintenance agreements are therefore required in all cases where the owner is other than the City of Loganville. A maintenance schedule should be part of the SMP facility plan. A maintenance agreement which specifies the frequency of maintenance and which alerts the City of such maintenance activities is required. The City will maintain an inventory of privately maintained SMP facilities, inspections and performance of required maintenance. The
City requires at a minimum a yearly inspection report on privately maintained SMP facilities. The City requires stormwater management easements around all SMP/ stormwater management facilities.

9.0 Limitations
The purpose of this manual is to provide general design guidance with respect to standard engineering design practices and procedures. It is the responsibility of the person responsible-in-charge for design to verify the engineering calculations through appropriate engineering design references. Any comments or questions concerning this manual should be directed to the Public Works Stormwater Management Department.

STATE OF GEORGIA CITY OF LOGANVILLE
Maintenance Agreement

WHEREAS, the Property Owner or Homeowner Association
____
recognizes that the wet or extended detention facility or facilities (hereinafter referred to as "the facility" or "facilities") must be maintained for the development called, ________________________, located in Land Lot(s) ______,District(s) _______ , of the City of Loganville, Walton County, Georgia; and,

WHEREAS, the Property Owner or Homeowner Association is the owner of real property more particularly
described on the attached Exhibit A as recorded by deed in the records of the Clerk of Superior Court of Walton County in Deed Book _____________ at Page(s) _________ (hereinafter referred to as "the Property"), and,

WHEREAS, The City of Loganville (hereinafter referred to as "the City") and the Property Owner or Homeowner Association, or its administrators, executors, successors, heirs, or assigns, agree
that the health, safety and welfare of the citizens of the City require that the facilities be constructed and maintained on the property, and,

WHEREAS, the Development Regulations require that facility or facilities as shown on the approved development plans and specifications be constructed and maintained by the Property Owner or Homeowner Association, its administrators, executors, successors, heirs, or assigns.

NOW, THEREFORE, in consideration of the foregoing premises, the mutual covenants contained herein, and the following terms and conditions, the parties hereto agree as follows:

SECTION 1.
The facility or facilities shall be constructed by the Property Owner or Homeowner Association in accordance with the plans and specifications for the development.

SECTION 2.
The Property Owner or Homeowner Association, its administrators, executors, successors, heirs or assigns shall maintain the facility or facilities in good working condition acceptable to the City and in accordance with the schedule of long term maintenance activities agreed hereto and attached as Exhibit B.

SECTION 3.
The Property Owner or Homeowner Association, its administrators, executors, successors, heirs or assigns hereby grants permission to the City, its authorized agents and employees, to enter upon the property and to inspect the facilities whenever the City deems necessary. Whenever possible, the City shall provide notice prior to entry. The Property Owner or Homeowner Association shall execute a twenty five (25) foot public access easement in favor of the City of Loganville to allow the City to inspect, observe, maintain, and repair the facility as deemed necessary. A fully executed original easement is attached to this Agreement as Exhibit C and by reference made a part hereof.

SECTION 4.
In the event the Property Owner or Homeowner Association, its administrators, executors, successors, heirs or assigns fails to maintain the facility or facilities as shown on the approved plans and specifications in good working order acceptable to the City and in accordance with the maintenance schedule incorporated in this Agreement, the City, with due notice, may enter the property and take whatever steps it deems necessary to return the facility or facilities to good working order. This provision shall not be construed to allow the City to erect any structure of a permanent nature on the property. It is expressly understood and agreed that the City is under no obligation to maintain or repair the facility or facilities and in no event shall this Agreement be construed to impose any such obligation on the City.

SECTION 5.
In the event the City, pursuant to the Agreement, performs work of any nature, or expends any funds in the performance of said work for labor, use of equipment, supplies, materials, and the like, the Property Owner or Homeowner Association shall reimburse the City, or shall forfeit any required bond upon demand within thirty (30) days of receipt thereof for all the costs incurred by the City hereunder. If not paid within the prescribed time period, the City shall secure a lien against the real property in the amount of such costs. The actions described in this section are in addition to and not in lieu of any and all legal remedies available to the City as a result of the Property Owner or Homeowner Association's failure to maintain the facility or facilities.

SECTION 6.
It is the intent of this agreement to insure the proper maintenance of the facility or facilities by the Property Owner or Homeowner Association; provided, however, that this Agreement shall not be deemed to create or effect any additional liability of any party for damage alleged to result from or caused by stormwater runoff.

SECTION 7.
Sediment accumulation resulting from the normal operation of the facility or facilities will be catered for. The Property Owner or Homeowner Association will make accommodation for the removal and disposal of all accumulated sediments. Disposal will be provided onsite in a reserved area(s) or will be removed from the site. Reserved area(s) shall be sufficient to accommodate for a minimum of two dredging cycles.

SECTION 8.
The Property Owner or Homeowner Association shall provide the City with a bond or a letter of credit providing for the maintenance of the facility or facilities for a period of not less than ten years from the date of execution of this Agreement. The bond or letter of credit shall be in

the amount of fifty percent (50%) of the construction costs of the facility or facilities pursuant to Section 8.7.5 of the City's Development Regulations concerning Maintenance Agreements-A copy of the bond or letter of credit is attached to this Agreement as Exhibit D and by reference made a part thereof.

SECTION 9.
The Property Owner or Homeowner Association shall use the standard SMP Operation and Maintenance Inspection Report attached to this agreement as Exhibit E and by this reference made a part hereof for the purpose of a minimal annual inspection of the facility or facilities by a qualified inspector.

SECTION 10.
The Property Owner or Homeowner Association, its administrators, executors, successors, heirs and assigns hereby indemnifies and holds harmless the City and its authorized agents and employees for any and all damages, accidents, casualties, occurrences or claims which might arise or be asserted against the City from the construction, presence, existence or maintenance of the facility or facilities by the Property Owner or Homeowner Association or the City. In the event a claim is asserted against the City, its authorized agents or employees, the City shall promptly notify the Property Owner or Homeowner Association and the Property Owner or Homeowner Association shall defend at its own expense any suit based on such claim. If any judgment or claims against the City, its authorized agents or employees shall be allowed, the Property Owner or Homeowner Association shall pay for all costs and expenses in connection herewith.

SECTION 11.
This Agreement shall be recorded among the deed records of the Clerk of Superior Court of Walton County and shall constitute a covenant running with the land and shall be binding on the Property Owner or Homeowner Association, its administrators, executors, heirs, assigns and any other successors in interest.

SECTION 12.
This Agreement may be enforced by proceedings at law or in equity by or against the parties hereto and their respective successors in interest.

SECTION 13.
Invalidation of any one of the provisions of this Agreement shall in no way effect any other provisions and all other provisions shall remain in full force and effect.

MAINTENANCE AGREEMENT
SO AGREED this day of __________________________ .

PROPERTY OWNER OR HOMEOWNER ASSOCIATION

BY: Attest:

Title: Title:

Approved as to form:

By: Date:

City Attorney

LOGANVILLE, GEORGIA

Attest: By:
City Clerk

City Manager

(SEAL)

Attachments: Exhibit A (Plat and Legal Description)
Exhibit B (Maintenance and Inspection Schedule)
Exhibit C (Access Easement)
Exhibit D (Standard SMP Operation and Maintenance Inspection Report)

CITY OF LOGANVILLE
SMP Facility Operation and Maintenance Inspection Report for Pond Facilities
(THIS MAY BE USED AS A TEMPLATE FOR OTHER SMPs)

Inspector Name

Community

Inspection Date

Address

Type of SMP

Watershed Tax Map

CHECKED MAINTENANCE ITEM INSPECTED

Yes Reqd.

Not Regd.

OBSERVATIONS & REMARKS

I. Pond Facilities

A. Pond Dam Embankments and Emergency Spillways

1. Vegetation and Ground Cover Adequate

2. Surface Erosion

3. Animal Burrows

4. Unauthorized Planting

5. Cracking, Bulging, or Sliding of Dam

a. Upstream Face

b. Downstream Face
c. At or Beyond Toe

d. Emergency Spillway

6. Pond, Toe,& Chimney Drains

7. Clear & Funct.

8. Seeps/Leaks on Downstream Face

9. Slope Protection or Riprap Failures

10. Vertical and Horizontal
Alignment of Top of Dam as
Per "As-built" Plans.

11. Emergency Spillway Clear of
Obstructions and Debris.

12. Other (Specify)

B. Riser and Principal Spillway
Type:
Reinforced Concrete

Corrugated Pipe

Masonry

* Indicates Dry Ponds Only
1. *Low Flow Orifice Obstructed

2. *Low Flow Trash Rack

a. Debris Removal Necessary

b. Corrosion Control

3. Weir Trash Rack Maintenance

a. Debris Removal Necessary

b. Corrosion Control

4. Excessive Sediment
Accumulation Inside Rider

5. Concrete/Masonry Condition

Riser & Barrels

a. Cracks or Displacement

b. Minor Spalling (<1*)

c. Major Spalling (Rebars Exposed)

d. Joint Failures

e. Water Tightness

6. Metal Pipe Condition

7. Control Valve

a. Operational/Exercised

b. Chained and Locked

8. Pond Drain Valve

a. Operational/Exercised

b. Chained and Locked

9. Outfall channels

10. Functioning

11. Other (Specify)

C. Permanent

Poll – Wet Ponds

1. Undesirable Vegetative
Growth

2. Floating or Floatable Debris
Removal Required

3. Visible Pollution